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The Municipal Building, New York City; An Office Building for the City Government.

The new city office building of New York, known as the Municipal Building, will be much the largest municipal administration building in existence, and will rank with the largest privately built office buildings. It is interesting both as a municipal enterprise and as a notable piece of construction. About two weeks ago its steel framework was erected to the top of the tower. The photograph herewith, taken from the editorial offices of Engineering News on the 19th floor of the St. Paul Building, about 1,000 ft. southwesterly of the new structure, shows this stage of completion. Following old custom, the attainment of the peak of the building by the framing was marked by hoisting a flag at the highest beam level, 540 ft. above curb. Meanwhile, the middle portion of the height of the main body of the building is receiving the enclosing skin of granite-faced brickwork, and in the interior the setting of the tile-arch floors is almost completed.

This 25-story structure will have nearly seven-eighths of an acre utilized space on each floor. Its extreme horizontal dimensions in the main shaft are approximately 380 × 166 ft., the plan being roughly triangular; the gross sectional area of the shaft is 42,686 sq. ft. The tower over the central portion is, of course, chiefly ornamental, and adds relatively little to the floor space (18,000 sq. ft., net, total for the tower, against 39,000 sq. ft. for one of the main floors). It is an important element in the framing, however, as its weight and the loads corresponding to wind moments are carried by the columns of the central part of the building, just where a wide arcade (53 ft. 5 ins. high, 35 ft. wide over roadway, 76 ft. 4 ins. wide over sidewalks) passes through on the line of Chambers St. The floor framing of the building is of very simple construction in general, but the columns in certain sections are of unusual size, partly because of the tower loads and partly because of the long span where Chambers St. is carried through the building. In addition, there is further complication in the lower four stories because of regrouping of the columns, those above being fitted to the general floor plan and those below being placed to suit the arcade arrangement and the caisson locations, etc. On account of this shifting of columns, some very heavy girder and truss work occurs in the fourth and fifth stories.

VIEW OF THE MUNICIPAL BUILDING, NEW YORK CITY, JULY 5, 1911.
(Highest tier of steelwork in place, 540 ft. above curb.)

The building will be 580 ft. high from curb level to the top of the figure on the tower; of this, 337 ft. is the height to the cornice (25 stories), while the height to the 34th floor, the highest used for offices, is 462 ft. The upper tiers of beams in our photograph is about 20 ft. below the base of the figure and 40 ft. below its top. Counting the depth below curb (145½ ft.), the height from base of deepest foundation pier to top of figure will be 725 feet.

That the column sections are correspondingly heavy is indicated by the maximum column load, 6,529,000 lbs., and the maximum section, 521.5 sq. ins. This section is made up of eight 8 × 6 × 1-in. angles, twelve 20 × 11/16-in. plates, and eight 36 × ⅞-in. plates. The total steel in the building amounts to about 26,000 tons. The granite wall facing totals some 600,000 cu. ft.

As will be remembered from our article of Nov. 17, 1910, p. 523-529, describing the foundations of the Municipal Building, exceptionally difficult work was required in order to secure a satisfactory foundation. The entire building is founded on pneumatic caisson piers, about two-thirds of which were sunk to bed-rock through 100 to 120 ft. of sand and gravel (bed-rock lying 130 to 150 ft. below street level). The northerly one-third of the building (in the accompanying view the farther wing) has its foundation caissons carried only to 70 ft. below street level, where they are footed on gravel; rock in this section of the site was at depths up to 200 ft. below street level, so that pneumatic sinking to rock was hardly feasible.

It happens that our picture shows quite a group of municipal and governmental buildings. In the foreground at the left is the Federal Post-Office, standing on a portion of City Hall Park, loaned to the United States by the city. Behind the post-office can be seen the graceful tower of the old City Hall, a building much praised by architects, and in spite of its great age even now an attractive feature of downtown New York.

Directly behind City Hall the County Court House, often called the Tweed Court House, shows in the picture, and to the right of this an ancient building also used for court purposes. The new Hall of Records is to the north of this old building, facing the west side of the Municipal Building's north wing. In the block lying west of the Hall of Records several large buildings (privately owned) are filled with offices of city departments and bureaus. Another large group of city offices is housed in the Park Row Building, showing in the near foreground at the right, for a long time the tallest building in New York and in the world. All these offices, it is expected, will be shifted to the new Municipal Building as soon as that is completed.

It was thought at first that the space afforded in the new building would be beyond the needs of the city. We understand, however, that as the estimates of required space have been received, revised and compiled, they have reached the point not only of taking up the entire building, but even exceeding its capacity. The offices which will be moved to the new building now occupy only 40% the space available in the latter, but many of the bureaus are quite cramped in their present quarters. It is fairly certain that no space in the new building will be available for renting to private tenants.

The available floor area of the Municipal Building will amount to about 700,000 sq. ft. The average rental paid now by the offices which will occupy it is not far from $1.75 per sq. ft. At this rate the new structure represents an annual rent

value of more than $1,230,000, which is enough to pay 8% on an investment of $15,400,000, or 10% on an investment of $12,300,000.

The total cost of the building is estimated at $12,000,000. About $2,000,000 was spent for foundations, while the superstructure contract (without interior finish or elevators) is for some $6,000,000.

The Brooklyn Bridge elevated terminal house appears in the picture immediately in front of the Municipal Building. One of the earliest projects for constructing the building contemplated carrying this terminal over into the second floor of the new building and providing there a joint terminal for the Brooklyn Bridge trains and the Third Avenue Elevated trains, whose present terminal is immediately to the right of the Municipal Building. Subsequent plans, however, arranged for a train station in the basement story of the buildong, and the bridge trains may ultimately be sent to this station by subway inclines. The station is primarily a part of the Centre St. loop subway, which extends northerly from this point to connection with the Manhattan and Williamsburg Bridges. The earning value represented by this station should be added to the above-given estimate of office-rent value to get the total income-producing power of the building.

The design and construction of the Municipal Building are in charge of the Department of Bridges of New York City, Mr. Arthur J. O'Keeffe, Commissioner. The architects are McKim, Mead & White. The general contractors are The Foundation Co. (caisson foundations) and the Thompson-Starrett Co. (superstructure).

The Influence of Forests on Stream Flow in the Merrimac River Basin, New Hampshire and Massachusetts.

After a considerable lull in the animated controversy over the relation between forests and stream flow, there now appears a study of this subject pertaining to the Merrimac River.

We give considerable space to the study; but more on account of its value as a long time record of stream flow alone than as a renewal of the controversy just mentioned. The stream-flow record included in the paper is a remarkably long one and probably as reliable as any in existence. The precipitation data for a few points in and near the river basin cover a much longer period than the run-off records, but for the drainage basin as a whole the rainfall records which have been assumed to be representative in character are for a slightly shorter period than the gagings of stream flow. The forestry data, as will be seen from what follows, are relatively meager, although the author of the paper appears to be reasonably well satisfied with them as a basis for his general conclusions.

The paper to which we refer was prepared by Lieut.-Col. Edward Burr, Corps of Engineers, U. S. Army, and is embodied in Document No. 9, 62d Congress, First Session, U. S. House of Representatives. The paper or report is dated May 23, 1910, and follows a very brief report on the advisability of improving the navigation of the Merrimac River between Haverhill and Lowell, Mass.

From the opening paragraph of Col. Burr's report on forestry and run-off, it appears that in November, 1908, the Chief of Engineers, U. S. Army, instructed

> all district officers to make the necessary investigations to determine what, if any, influence forests exert upon stream flow in the important navigable rivers in their districts and to furnish all data that tend to support or disprove the popular impression that forests diminish the height or frequency of freshets and increase the low-water flow of navigable rivers.

Col. Burr states that the collection and analysis of data on rainfall and run-off were made by Junior Engineer F. S. Stevens, while data on the drainage basin in general, and forestry conditions therein, were collected by Mr. H. A. Edgecomb (Clerk, U. S. Engineer Office). Col. Burr considered it pertinent to remark that Mr. Stevens

> undertook and completed the work in ignorance of the forestry data of the basin and with a strong predisposition in favor of the popular belief that forests do exert a material beneficial influence upon stream flow. The conclusions stated in his report are, therefore, to be accepted as the opinions of an observer heretofore biased in favor of that belief, and, in so far as they fail of agreement with it, may be accepted as being at least impartial.

As many of our readers know, the development of water power at Lowell, Mass., was begun in 1821 by the Merrimac Mfg. Co. Power development at Lawrence, a few miles below Lowell, was begun in 1845. The late Mr. James B. Francis was connected with the Lowell development from 1834 to his death in 1893, first as Assistant Engineer and then as engineer. Mr. Hiram F. Mills has been engineer of the Essex Co., of Lawrence, since 1869, and Engineer of the power company at Lowell since the death of Mr. Francis in 1893. Ever since 1849 a daily record has been kept of the stage of the Merrimac River, as shown by a gage below the dam at Lawrence. Col. Burr states that no change "in the regimen of the river that could have affected the relation between river stages and gage heights" at the location of the gage has taken place during the 60 years. Other records at Lawrence, which it is unnecessary to mention here, are also available.

Precipitation records for the Merrimac drainage area are available at Lowell since 1826 and for a duplicate gage at Lowell since 1856. Precipitation records are also available at Concord, Lakeport and the Weirs, N. H., since 1857 and 1858. More numerous, but not always complete, rainfall records for other stations in the basin are available since about 1886. The relative deficiency of rainfall records is in part made good by a close agreement between such records as exist for the period since 1886 throughout the whole drainage basin and records at two stations near the center of the drainage area, namely, at Concord and Lakeport, N. H.

After stating that throughout his paper "all theorizing has been avoided," the author gives a general summary of the conclusions drawn from the studies presented in the report. In order that our readers may not be held in suspense, we quote at once these conclusions of Col. Burr as follows:

> To the writer it seems clear that nothing in the following facts as a whole gives any support to the general popular belief that deforestation has exerted a harmful, and reforestation, to the extent of 25% of the entire area, has exerted a beneficial effect upon stream flow, in so far as concerns the Merrimac basin.
>
> On the contrary, there are larger indications in opposition to this belief than there are in support of it, but they are not sufficiently pronounced to warrant a definite conclusion to that effect, and may be obliterated by an extension of the study 20 years or more into the future.
>
> One positive and definite conclusion may be drawn from this study. Any attempt to determine general tendencies in this or a related subject on the basis of records covering periods of but 20 or 30 years is futile, and a period of more than 60 years, with complete and accurate records throughout, is necessary to conclusions of unquestionable accuracy. The characteristics of stream flow are so largely, if not so entirely, dependent upon precipitation, modified by temperature or other climatic influences, that any analysis of the former must be based upon a substantial elimination of the variations in the latter for the determination of the effect, if any, of variations in all other influences and the most casual examination of extended records of precipitation, of which a few are inserted hereafter, shows them to run through irregular cycles, which may, and in the general case do, extend over a period of 80 years or more.

The entire drainage area of the Merrimac River (Fig. 1) is 5,015 sq. mi. Of this, 3,815 sq. mi. are in New Hampshire and 1,200 in Massachusetts. The portion of the drainage area above Lawrence, which alone is considered in the paper, is 4,664 sq. mi. The elevation of the drainage area ranges from sea level to a maximum of some 5,000 ft., the headwaters lying at a general elevation of about 2,000 ft. In general, the "topographical conditions are conducive to rapid run-off, at least from the New Hampshire part of the basin, when climatic developments are favorable to freshet stages." The numerous lakes and ponds in the drainage area, the largest of which, Lake Winnepesaukee, has an area of 100 sq. mi., afford comparatively little effective storage. Considerable quantities of water are stored in the Massachusetts section and withdrawn entirely from the drainage area for the supply of the Boston Metropolitan Water District.

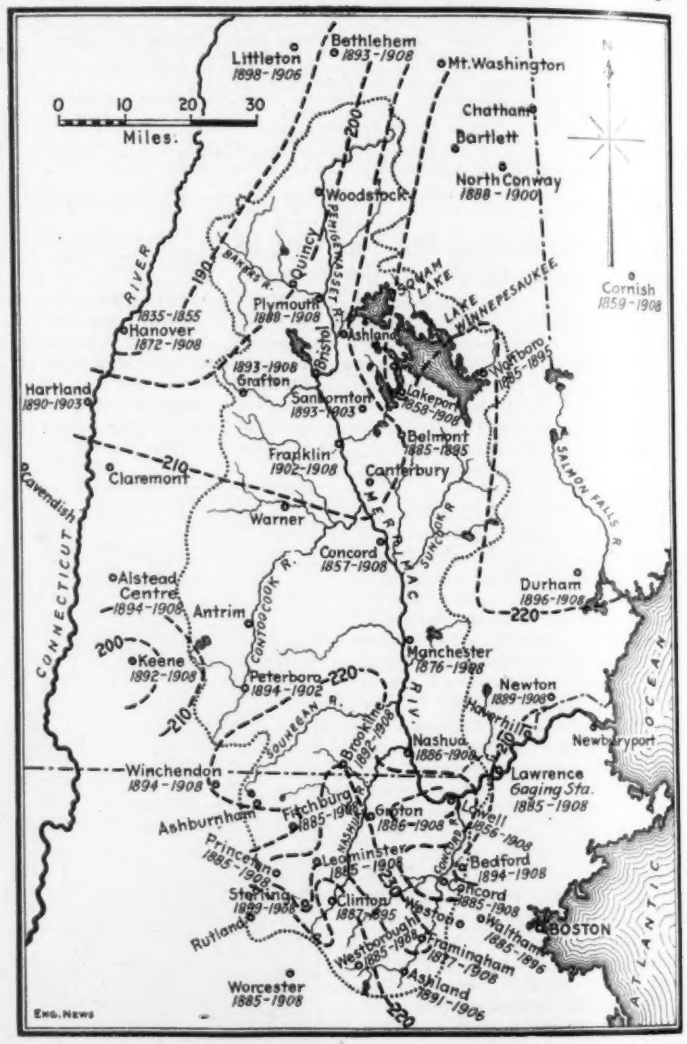


FIG. 1. DRAINAGE BASIN OF THE MERRIMAC RIVER ABOVE LAWRENCE, MASS., WITH LOCATION OF PRECIPITATION STATIONS.

(The length of the precipitation records at the various stations is given and also isohyetal lines for the five-year period, 1901-5. The mean precipitation for the whole basin and period was 214 ins. We have combined in one the two maps of the original report.—Ed.)

Forest Conditions.

The principal information available regarding forest conditions is to be found in the report of the United States Census for 1900, in reports of the New Hampshire Forestry Commission for 1904 and 1906 and in unpublished returns collected

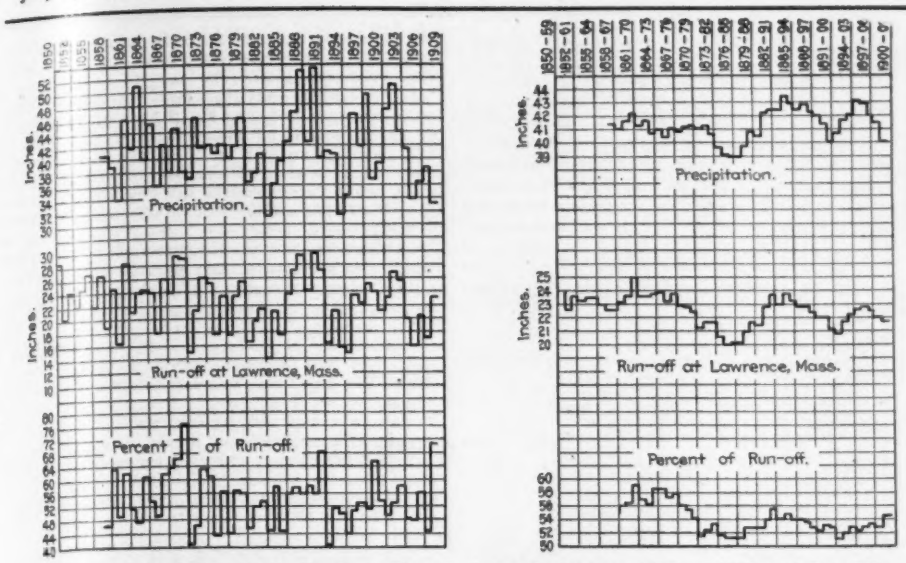


Fig. 2. Precipitation in the Merrimac River Drainage Basin from 1858 to 1908 and Run-Off At Lawrence, Mass., 1850 to 1908, Inclusive.

(The precipitation figures are the average of the rainfall records at Concord, N. H., and Lakeport, N. H. The "progressive" averages for both rainfall and run-off for any given year are the average of the 10-year period beginning with that year. We have combined in one diagram the annual and the progressive-period curves shown by two diagrams in the original report.)

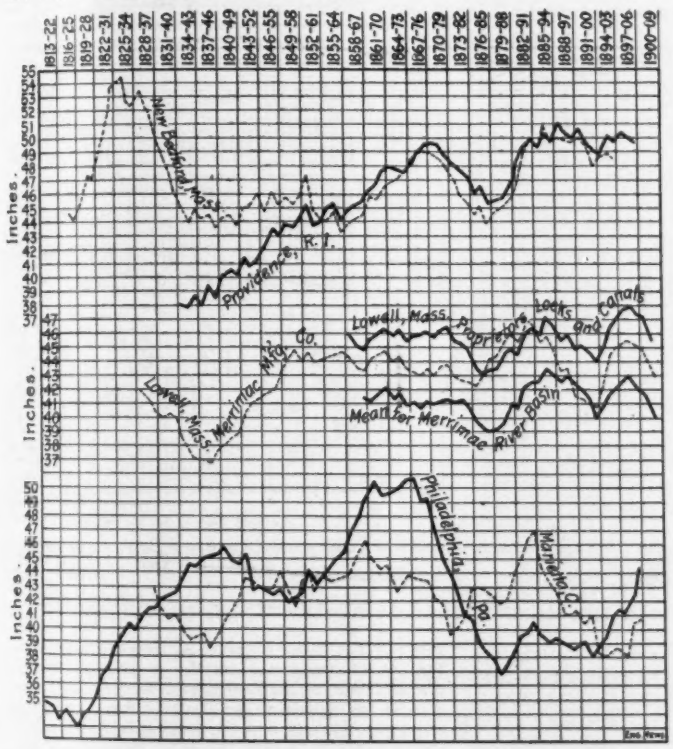


Fig. 3. Progressive 10-Year Averages of Annual Precipitation at Six Long-Record Stations: Lowell, Mass., (2 Stations), New Bedford, Mass., Providence, R. I., Philadelphia, Pa., and Marietta, Ohio.

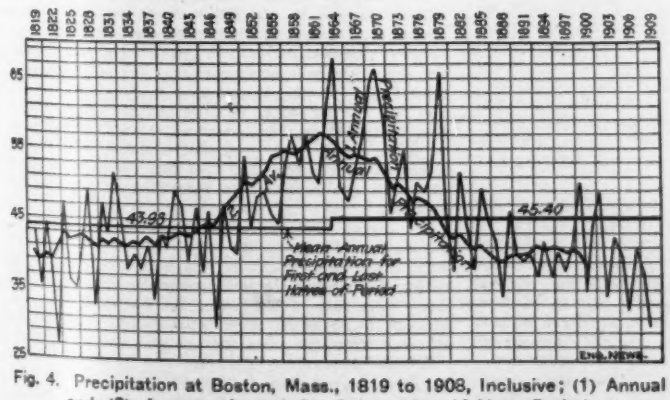


Fig. 4. Precipitation at Boston, Mass., 1819 to 1908, Inclusive; (1) Annual and (2) Average Annual for Progressive 10-Year Periods.

FIGS. 2 TO 4. DIAGRAMS SHOWING PRECIPITATION OR RAINFALL IN THE MERRIMAC RIVER DRAINAGE AREA AND LONG-TIME RECORDS FOR VARIOUS CITIES.

by Massachusetts towns in 1905 for an agricultural census of that state. Mr. Edgecomb had access to the Massachusetts figures. In addition to information bearing directly upon forested areas, use was made of census statistics showing the areas of improved and unimproved lands, and from these deductions were made as to forestry conditions. For the census years 1870 and 1880, unimproved lands reported were classified into woodland and other unimproved land. Col. Burr states:

> From these statistics, aided by the local information mentioned above, there have been derived data as to forested areas at 10-year intervals from 1850 to 1900 that suffice, for the purposes of this study, to determine approximately the changes that have occurred in the last 60 years.

The most essential forestry data available from actual records and deductions are summarized by Col. Burr in the table at the top of the next column.

It will be noted that these figures relate to New Hampshire alone, and to the whole state, rather than being confined to the Merrimac River drainage area.

Mr. Edgecomb presents in some detail such forestry data as he could find, and shows how he arrived at his conclusions as to the figures applicable to the Merrimac drainage area above Lawrence located in the two states of Massachusetts and New Hampshire. It seems unnecessary to take up Mr. Edgecomb's original data further than to state that the New Hampshire forestry surveys mentioned above afforded more or less

FOREST AREAS IN NEW HAMPSHIRE.

	Acres.	Percentage of State.	Percentage change by decades.
1830	2,600,464	45.1	....
1860	2,484,918	43.1	— 2.0
1870	2,517,465	43.7	+ 0.6
1880	2,651,725	46.0	+ 2.3
1890	3,232,450	56.1	+13.1
1900	3,882,958	67.4	+ 8.3

definite information regarding recent forested and unforested areas, while the 1905 census returns from Massachusetts towns already mentioned served the same general purpose for the Massachusetts portion of the drainage area. Due consideration was given to such collateral information, some of which was largely in the nature of impressions or opinions, as was available.

The significant conclusions as to forestry conditions in the Merrimac drainage basin drawn by Col. Burr are expressed in the following sentence:

> Deforestation continued progressively from the earliest settlements to about 1860-1870; reforestation through natural processes has progressed since 1870 at a rate which has increased from 1880 to 1900 or later; and forest areas are larger now than in 1800-1870 by as much as 25 to 30% of the entire basin.

The reports of the New Hampshire Forestry Commission for 1904 and 1906 contain estimates to the effect that 85% of that portion of the Merrimac drainage area above Plymouth, N. H., is in heavy forests and that of the southern part of the basin in the state 66% is forested. This gives an average of 70.5% for the portion of the basin in New Hampshire, while the Massachusetts agricultural census returns of 1905 show 35.6% of the Massachusetts portion of the basin in woodland. From these figures it is concluded that 62% of the entire Merrimac drainage basin is now in forest. It is also concluded that for some years past natural reforestation has been proceeding at the rate of about 1% in Massachusetts and about 1.1% in New Hampshire.

Perhaps it should be brought out that what constitutes forested areas is interpreted broadly (or perhaps we should say *thinly*) by Mr. Edgecomb and that his interpretation seems to be accepted without question by Col. Burr. Thus we find in Mr. Edgecomb's report a statement that the present forest stand in New Hampshire is in rare instances or localities more than 80 years of age; that below this "all ages are well represented," and that near the older settlements, particularly in the southeastern part of the state, "most of the woodland has been cut over two or three times, and sapling stands are more abundant than elsewhere." In extracts from the New Hampshire forestry reports cited by Mr. Edgecomb, we find such statements as these:

> The present forest consists entirely of second growth, chiefly in small lots, but occupying about two-thirds of the total land area. The forest area has been greatly extended by the abandonment of improved farm land, in rapid progress since 1880. . . .
>
> At least 800,000 acres of improved land have been abandoned since 1880, and a large part of this area is not yet fully stocked with trees.

In the section on Massachusetts conditions, Mr. Edgecomb states that "woodland includes forests over 30 years growth, forests under 30 years growth, seeds or transplanted seedlings, scrub and unclassified woodland." Elsewhere, Mr. Edgecomb cites the state forester of Massachusetts to the effect that forests which have been run through by fires are seldom completely destroyed, and then adds

> a portion of the forest remains, and the destroyed section, without artificial means, soon grows to brush or scrub, effectively covering the soil and retaining its original character so far as it has any regulative influence on stream flow.

Precipitation.

Col. Burr states that the Massachusetts section of the Merrimac drainage basin

> is fairly well covered by the [precipitation] records since 1885, but that there are relatively few records for New Hampshire, and none within the basin for the 900 sq. mi. above Plymouth, or for the large area on the western slope of the basin between Peterboro and Grafton, a distance of 45 mi.

The only records going back of 1885 are as follows: Lowell, Mass., 1826 to 1908, and also for another gage, 1856 to 1908; Concord, N. H., 1857 to 1908; Lakeport, N. H., 1858 to 1908; the Weirs, N. H., 1858 to 1905; Manchester, N. H., 1876 to 1908; Framingham, Mass., 1877 to 1908.

The investigators appear to have spared no

effort to collect as complete rainfall data as possible. The report contains transcripts of not only the long-time records named, but also of many short-time records, and the investigators went sufficiently far afield to bring in various records from stations in Vermont and also in New Hampshire and Massachusetts outside the drainage area. The many gaps in the records, particularly in the short-time records, were painstakingly filled by carefully-made estimates based on records for other stations which agreed

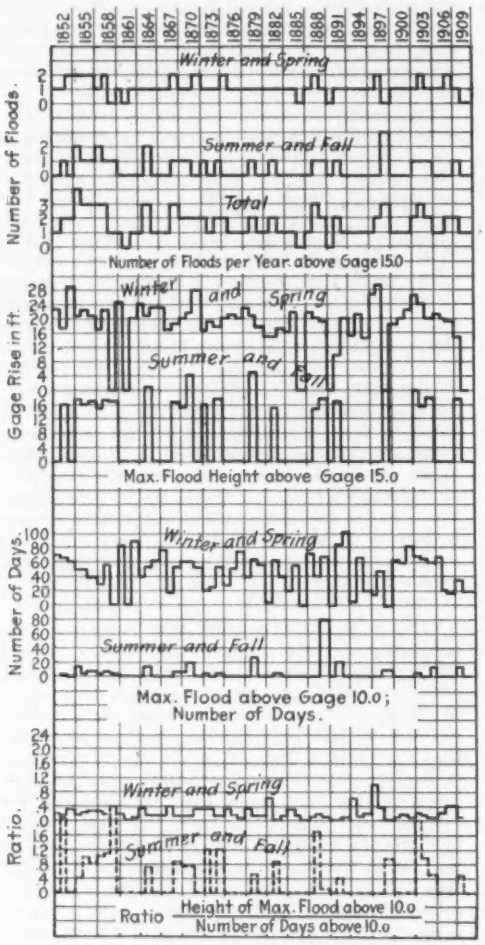

Number of Floods.
Winter and Spring
Summer and Fall
Total
Number of Floods per Year above Gage 15.0
Gage Rise in ft.
Winter and Spring
Summer and Fall
Max. Flood Height above Gage 15.0
Number of Days.
Winter and Spring
Summer and Fall
Max. Flood above Gage 10.0; Number of Days.
Ratio.
Winter and Spring
Summer and Fall
Ratio Height of Max. Flood above 10.0 / Number of Days above 10.0

Fig. 5. Actual Records by Years.

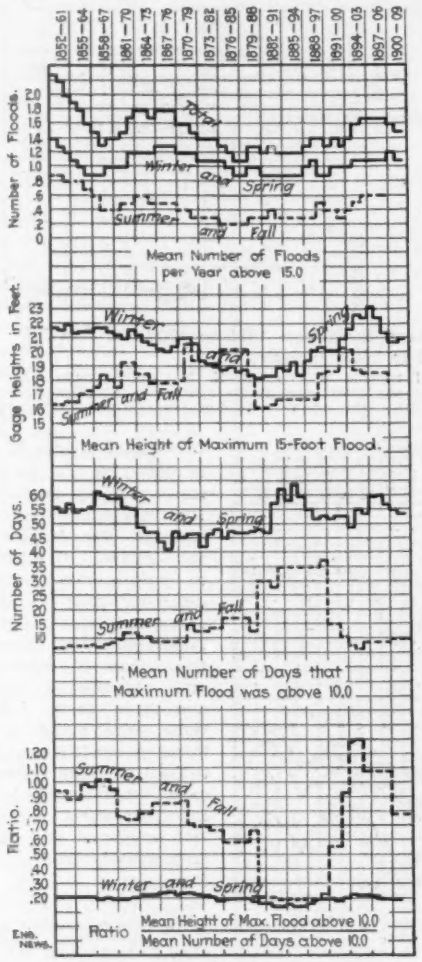

Number of Floods.
Total
Winter and Spring
Summer and Fall
Mean Number of Floods per Year above 15.0
Gage heights in Feet.
Winter and Spring
Summer and Fall
Mean Height of Maximum 15-Foot Flood.
Number of Days.
Winter and Spring
Summer and Fall
Mean Number of Days that Maximum Flood was above 10.0
Ratio.
Summer and Fall
Winter and Spring
Ratio Mean Height of Max. Flood above 10.0 / Mean Number of Days above 10.0
ENG. NEWS.

Fig. 6. Averages for Progressive 10-Year Periods.

FIGS. 5 AND 6. NUMBER, DURATION, AND GAGE-HEIGHT OF FLOODS.

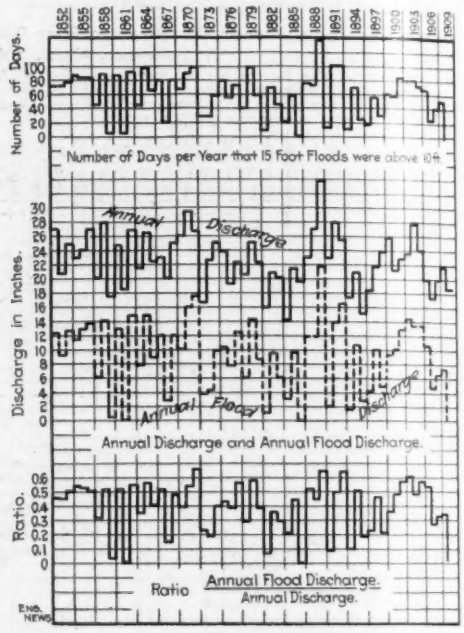

Number of Days.
Number of Days per Year that 15 Foot Floods were above 10.4
Discharge in Inches.
Annual Discharge
Annual Flood Discharge
Annual Discharge and Annual Flood Discharge.
Ratio.
Ratio Annual Flood Discharge. / Annual Discharge.
ENG. NEWS.

Fig. 7. Actual Records by Years.

FIGS. 7 AND 8. TOTAL DURATION OF FLOODS BY INCHES

FIGS. 5 TO 10. FLOODS, FLOOD AND ANNUAL DISCHARGE, AND LOW WATER IN THE MERRIMAC RIVER AT LAWRENCE, MASS., 1850-1908.

with those which were deficient. It is unnecessary to take these estimates into consideration further than to say that careful compilation of all available precipitation records in the drainage basin and a series of isohyetal maps for the drainage basin, made up of progressive five-year rainfall averages, indicated that the average for the whole basin from 1886 to 1908 was in close agreement with the average of the Concord and Lakeport precipitation for the same period. The differences between the averages for the whole basin and the Lakeport-Concord averages were only

0.64% in the mean of the 19 periods, a difference for single periods not exceeding 3.27% and individual differences of less than 1% for 12 periods, as used for all comparisons of precipitation and run-off, show no periodic difference greater than 1.8% and in nine out of 14 periods a difference less than 1%. For some single years an estimate based on the Lakeport-Concord average might be considerably in error and too much weight must not be given to the diagram of annual precipitation, but yearly irregularities tend to balance one another in the course of 10 years and estimates of the precipitation for the basin for 10-year periods based on the Lakeport-Concord averages are believed to approximate so nearly to the actual facts as to lead to no material error in deducing relative tendencies.

Accordingly, the average of the Concord and Lakeport figures from 1858 to 1908 was used as indicating the precipitation throughout the whole Merrimac River drainage area.

Col. Burr is of the opinion that the precipitation data used in the study are

as complete as those available for the drainage area of any other navigable stream in the United States, and are more complete than for the very large majority of such streams, and if a conclusion based on this data is not reasonably correct, then the study of such questions may as well be abandoned at this time so far as concerns streams of a size to be classed as navigable rivers.

The precipitation for the Merrimac River basin from 1858 to 1908 as thus deduced is shown diagrammatically by Fig. 2. The diagram gives both the annual figures and averages for progressive 10-year periods. For purposes of comparison, the precipitation records of two gages at Lowell, Mass., and at four other stations are shown by Fig. 3, while Fig. 4 shows a long-time record at Boston, Mass. We reprint Col. Burr's comments on the precipitation diagrams for the Merrimac basin as a whole and for the several stations just mentioned.

The results of these estimates for annual precipitation and for progressive 10-year periods are shown by diagrams (Fig. 2). Annual precipitation since 1858 has varied between 31 and 53 ins. and extremes are distributed throughout the record. It may be observed that the two great floods that occurred during this period did not fall in years of greatest annual precipitation, the records for 1895 and 1896 being 46 and 42 ins. The precipitation for 1895 has been exceeded in seven different years and although 10% greater than for 1896, the 1895 flood was lower by 2¼ ft. than the flood of 1896, the highest flood on record.

The diagram for progressive 10-year periods shows variations in 10-year means between 39 and 43 ins. or of about 10%. Beginning in 1858 there is for 15 years or more an irregular variation of about 3%, followed by a drop in 1876-1877 to the minimum of 39 ins. and by two cycles that reach the maximum of 43 ins. in 1884 and 1894, with a return in 1908 to 40 ins. The indications from this period of 50 years are that precipitation had reached its minimum in the middle seventies, coincident approximately with the time of minimum forest areas, but it is known that a period of great drought had occurred about 1840, and for a further study of this question, all available data have been collected for near-by New England stations prior to 1850.

On Fig. 3 are shown the records for progressive 10-year periods for Lowell (Merrimac Manufacturing Co.) since 1826, Lowell (Proprietors, Locks, and Canals) since 1836, Providence since 1832, and New Bedford since 1815; and on Fig. 4 are the annual and 10-year records for Boston since 1818. To Fig. 3 have also been added the 10-year records for Philadelphia, Pa., and Marietta, Ohio. Many points may be noted from these diagrams, of which none is more curious than the close parallelism of the records for Providence and New Bedford (30 mi. apart) since 1850 and the wide divergence between them for earlier years.

The diagrams show marked differences in the cyclic variations in precipitation from place to place, but none of them offers any support to the theory or contention that deforestation decreases precipitation and vice versa. On the contrary, the best indications of these diagrams are that, so far as concerns the Merrimac basin and southeastern New England, as well as Eastern Pennsylvania and Ohio, more rain has fallen as forests have decreased.

In this connection especial attention is invited to the diagrams of precipitation at Boston since 1818, which illustrate many points. The variation in annual precipitation has been extreme, the minimum being 27 ins. and the maximum 67 ins. The highest records fall between 1862 and 1878, while the lowest are found in the first and last thirds. The third diagram [lower curve, Fig. 4] illustrates the futility of averages for half periods, since the two averages differ by only 3% and give no indications of the marked cycle between 1849 and 1887. The diagram for progressive 10-year periods shows a slight increase to 1823, followed by minor changes to 1840; then comes a gradual increase of about 30% to 1861, and a similar decline to 1887, with a drop in 1908 to about the minimum of 1819.

This diagram shows the worthlessness of short-time climatic records in discussing progressive changes and tendencies. The record for the 50 years since 1800 would indicate a radical decrease in precipitation, but the record since 1818 tells another story. The other long-time precipitation records diagrammed on Fig. 3 make plain also that discussions of climatic changes and of stream flow controlled by them, based on records covering even as much as 50 years may, and doubtless will, lead to erroneous conclusions, and that data for at least 80 or 100 years are necessary to approximately accurate deductions.

The Boston diagram for the same reasons shows the fallibility and infallibility of the "oldest inhabitant" who, if his memory carried him back accurately for 35 to 50 years, would correctly remember that there was much more rain in his youth, but who, if it took him back 65 to 85 years, might equally well remember no difference from the rains and the snows of his boyhood.

In some discussions on stream flow so much reliance has been placed upon the recollections of elderly men regarding the conditions of wells, springs and brooks during their early years that it is worth noting that major climatic cycles extend much beyond the life of the average man or the accurate recollections of octogenarians.

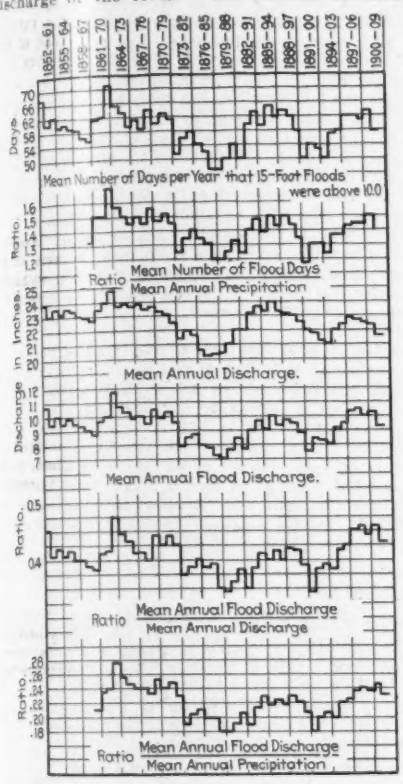


Fig. 8. Averages for Progressive 10-Year Periods.

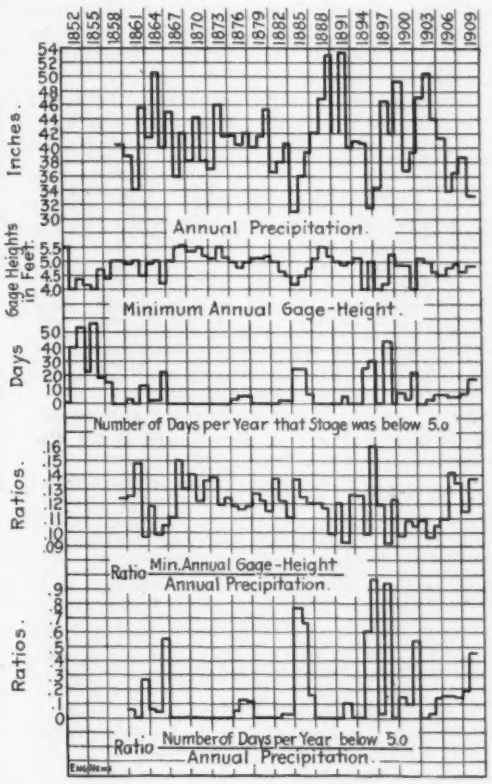


Fig. 9. Actual Records by Years.

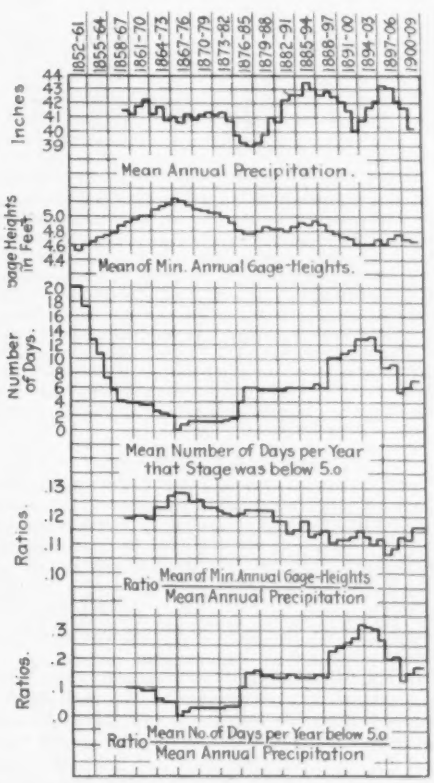


Fig. 10. Averages for Progressive 10-Year Periods.

FIGS. 9 AND 10. GAGE-HEIGHT AND DURATION OF LOW-WATER STAGES AND PRECIPITATION.

AND TOTAL ANNUAL AND FLOOD DISCHARGES OF DEPTH.

AND PRECIPITATION IN THE DRAINAGE BASIN, 1850-1908, BOTH ANNUALLY AND BY AVERAGES FOR PROGRESSIVE TEN-YEAR PERIODS.

Run-Off.

Col. Burr's remarks on run-off data are prefaced by the statement that the power companies at Lowell and Lawrence acquired control of the outlets to Winnepesaukee, Swan and other lakes in the Merrimac drainage basin in 1846, and have since utilized the lakes as storage reservoirs for the benefit of their water-power plants. The Essex Co., of Lawrence, has kept a daily record of the height of the river below its dam since 1849. It has also made many measurements of the "discharge of the river over its dam and through its canals, wheels and tail races." These measurements have been used as the basis for a discharge or rating curve. Since 1888, Col. Burr states, the company has "determined the actual discharge of the river." All records mentioned were given careful consideration by Mr. Stevens in his detailed study of precipitation and run-off appended to Col. Burr's report. The run-off computations for the Merrimac River at Lawrence are characterized by Col. Burr as "of exceptional value compared with the summary data available for most navigable rivers."

To eliminate "erratic variations, with no relation one to another that are found in climatic records compiled from month to month or year to year and that occur in stream conditions controlled largely by such variations," there must be employed in any such study as this, Col. Burr states, "such a system of averages as best fits the case in hand." He adds that such averages merely facilitate a "comparison of facts" and that correct deductions can be expected in all cases only when averages are considered in connection with the facts themselves. To continue in Col. Burr's own language:

> In the arrangement of erratic records, the length of the periods over which averages are taken must be so selected as to control eccentric data that are merely incidental to such records and at the same time to show general tendencies and to provide results in numbers sufficient for comparison. It is, for instance, evident that an average covering the entire period of the record is valueless in illustrating tendencies and that averages by half periods are almost equally futile. In the present study a period of 10 years has been selected as being long enough to eliminate variations that are merely erratic from year to year.
>
> To form a continuous record, these 10-year periods are arranged progressively, each beginning one year later than the next preceding period, instead of consecutively as when each period begins with the year next following the last year of the preceding period. By the use of these periods, hereafter called "progressive 10-year periods," a continuous record from year to year is obtained, the beginning and end of each change is defined and the breaking up of irregular cycles, with resulting incorrect deductions, is avoided, but the labor involved in arranging the data has been multiplied tenfold.

The run-off data for the Merrimac River are presented by means of the two lower curves shown by Fig. 2, the lowest curve showing the percentage of run-off. Col. Burr remarks that the diagrams for precipitation and run-off "illustrate clearly the small value of annual statistics alone for ascertaining progressive tendencies in any field where results depend primarily upon data varying so erratically as do climatic records for short periods." The annual run-off from the Merrimac basin has ranged from 14 to 30 ins. actual and from 40 to 76% of the precipitation. It not infrequently happens that a large annual run-off is followed immediately by a small one. Large annual precipitations are not necessarily accompanied by large run-offs. The diagrams show that run-off is materially affected from year to year by factors other than the mere amount of annual precipitation."

By means of the 10-year progressive averages, a study of tendencies can be made. The curves for precipitation and run-off "are the same general shape and very nearly parallel," but the percentage run-off curve "fails to parallel closely that of precipitation, and its general tendency is downward." Quoting further from Col. Burr's discussion:

> The highest percentages are found between 1860 and 1873, when the precipitation was moderate, and are followed by a decline in both to the minimum of 1875-1885; but the two cycles of increased precipitation since 1885 have not been accompanied by corresponding increases in the percentage of run-off, and in recent years this percentage has been less than in 1860-1875, although the average precipitation has been greater.
>
> It is to be noted that the percentage of run-off is approximately the same in 1899-1908 as for 1858-1867; that the extreme variations of about 8% occurred between 1865 and 1885 when no marked change in forest areas took place; that the maximum percentage occurred when forest areas were a minimum; and that a progressive increase of 25 to 30% in forest areas since 1880 has not been accompanied by any corresponding change in percentage of run-off.
>
> The average percentage of run-off has been slightly less during the period of reforestation than during the period of maximum deforestation, but the difference is relatively small and variations in percentage have moved without any distinct relation to forest conditions so far as may be determined from this record of 60 years. Mr. Stevens in his report discusses the effect on total run-off of the storage capacity of the lakes in the basin and of the water withdrawn for the supply of the Boston Metropolitan District. The former may be neglected as having no effect in this connection since the stored water finally reaches the river. The latter represents water permanently withdrawn from the stream in slowly increasing quantities and may account in part for the lower percentages of run-off during recent years, but its amount is relatively so small as not to affect the deductions to be drawn from the data at hand.
>
> The average run-off in the Merrimac River varies with the average precipitation throughout its basin, and the percentage of precipitation that reaches the river is not appreciably affected by changes as great as 25 to 30% in forested areas. The increase in forested areas since 1880 appears to be accompanied by a slight decrease in percentage of run-off, but the tendency is not sufficiently marked to warrant a definite conclusion to that effect.

We also quote from Mr. Stevens' supplementary report a portion dealing directly with his run-off studies:

> Discharge quantities, as estimated by means of a curve, can, as a rule, be regarded only as very fair approximations. And, while in this case the measurements upon which the curve is based were undoubtedly of exceptional accuracy, the fact that the curve gives mean discharge for 24 hours on the authority of one gage reading, in a stream subject at times to rapid rises and falls, admits the possibility of an occasional important difference between the actual daily discharge and the discharge indicated by the curve. The discharge in second-feet for 24 hours, corresponding to the mean monthly gage height, was taken as the mean monthly discharge. The mean monthly gage height used was the mean for working days; and, since on Sundays and holidays during the low-water period the river at the gage is much lower and the discharge much less than on working days, the estimated discharge for the low-water months and consequently for the year, is somewhat greater than the actual. But, as the yearly difference is fairly uniform, and the difference for 10-year periods exceptionally so—and as this study is one of the relative rather than actual quantities, this error can hardly have any appreciable influence on final conclusions.

COMPARISONS OF ANNUAL DISCHARGE, AS ESTIMATED BY MEANS OF DISCHARGE CURVE, WITH THAT COMPUTED BY ESSEX CO.

Year.	Discharge in ins. as estimated by means of discharge curve.	Discharge in ins. from computations of Essex Co.	Difference.	Per cent. of difference.	Mean per cent. of difference for progressive 10-year periods, beginning with given date.
1888....	29.39	27.41	1.98	6.73	3.78
1889....	24.43	24.30	.13	.53	3.62
1890....	29.58	29.15	.43	1.45	4.23
1891....	27.24	26.66	.58	2.13	5.01
1892....	16.56	15.82	.74	4.48	5.14
1893....	21.16	19.55	1.61	7.60	4.85
1894....	15.78	15.28	.50	3.15	4.28
1895....	15.21	13.96	1.25	8.21	4.26
1896....	23.66	23.07	.59	2.49	3.99
1897....	22.43	22.19	.24	1.07	4.39
1898....	25.41	24.11	1.30	5.11	5.06
1899....	23.94	22.25	1.69	6.62	5.49
1900....	21.41	19.43	1.98	9.25	....
1901....	23.16	22.37	.79	3.42	....
1902....	27.07	26.65	.42	1.55	....
1903....	25.65	25.16	.49	1.91	....
1904....	20.09	19.50	.59	2.94	....
1905....	16.30	15.40	.90	5.52	....
1906....	20.54	19.21	1.33	6.49	....
1907....	17.34	15.99	1.35	7.79	....
1908....	23.47	21.26	2.21	9.40	....
Means..	22.37	21.37	1.00	....	4.50

All errors involved in the estimates of discharge are, with the exception of that due to over-estimating the discharge for Sundays and holidays, compensating in effect rather than accumulating; and the aggregate error is not of such a nature as to affect the form of the curves for 10-year periods. Table 4 gives the yearly discharge as actually measured by the Essex Co. for 21 years, 1888-1908, inclusive; and the discharge for the same years as estimated by the methods described above, showing that, while the difference for some years is rather large, for 10-year periods it is less than 5½% at its maximum, and varies less than 2% for the twelve 10-year periods compared.

Floods and Low Water.

There being no "danger line" shown by the Merrimac gage heights, it was concluded after a study of the continuous daily hydrograph of the river for about 60 years to "class as floods only such rises as reach or exceed a height of 15 ft. on the gage and to consider such a rise as constituting only one flood between the times it passed 8 ft. on the gage in rising and again in falling."

In computing flood durations and discharge, consideration was given only to the period during which the river was above gage 10 ft. Floods were also classified as "winter and spring," or those due to the spring rains and melting snow, and "summer and fall," due to rainfall modified by ground-water conditions. Nearly all the floods of extraordinary height occurred in the spring.

For convenience in comparison, we have brought together on two facing pages a half dozen diagrams which were presented on separate sheets in the original report. Figs. 5 and 6 are pertinent to the present discussion. They show the number, duration and heights of floods from 1850 to 1908, together with subclassifications into winter and spring and summer and fall floods. There is also a curve giving the ratio of the heights of floods above 10 ft. to the number of days that they were above that height. Fig. 6 shows the same curves for progressive 10-year periods.

Col. Burr's general comments on the portion of the diagrams bearing on the frequency of floods as shown by the progressive 10-year averages, Figs. 5 and 6, are as follows:

The diagrams (Fig. 6) of the average number of floods for 10-year periods indicate a maximum number of summer and fall floods at the beginning of the record (1850-1859), and a decrease to a minimum in 1876-1885, followed by an increase toward the end of the record. They show less variation for winter and spring floods, but with the maximum still between 1850 and 1859, a minimum in 1856-1865, a lesser maximum in 1866-1876, a decline again to the minimum, and a slight increase at the end of the record. The summary of all floods parallels fairly closely the record of summer and fall floods, with the minimum between 1876 and 1894, followed by an increase to 1906.

Bearing in mind that forest areas were least between 1880 and 1890 and that they have since increased by 25 to 30% of the entire basin, it certainly may be said that these diagrams show no increase in the frequency of floods with deforestation or decrease with material reforestation. On the contrary, if this record has any value at all it tends to show opposite effects. It may be noted also that it fails to show any relation between periods of maximum precipitation and of maximum flood frequency. There was undoubtedly a period in 1850-1800 when floods were more frequent than in later decades, but no great weight is necessarily to be given to the apparent increase in flood frequency prior to 1906. Floods, both as to their frequencies and as to their extreme heights, result from unusual combinations of climatic conditions, erratic as to their periods, and the increase in flood frequency to 1906 was perhaps in part due to the more frequent recurrence of such combinations rather than to the increase in forested areas.

Flood frequency has not decreased with the reforestation of 25 to 30% of the basin, nor did it increase during the last of the period of deforestation. The data available might be held to indicate contrary results, but the period from 1850 to 1908 is not sufficiently long to permit of a definite conclusion to that effect.

A sentence from Mr. Stevens' report may be inserted here:

To show with any accuracy what portions of such changes were due to deforestation, and what to other causes, would be practically impossible; and if any are detected they should be considered as resulting from the sum of the effect of all the elements concerned rather than from that of the forest element alone.

Furthermore, Mr. Stevens says that he feels "justified in making the statement that the amount of flood discharge has undergone no appreciable change since 1850."

Commenting in general terms upon the height and duration of maximum floods, Col. Burr says:

From the above it appears that extraordinary floods have resulted from exceptional climatic conditions irrespective of material changes in forested areas; that after 1850 the average maximum height of spring floods showed a tendency to decrease until about 1882 and the average duration to decrease until about 1872-1882, since which dates there have been in both height and duration a tendency toward increase; that there was a slight tendency toward increase in the maximum height of summer and fall floods from 1850 to 1908, with extreme variations between 1870 and 1890, and that there was a slight increase in the duration of these floods to about 1880, followed by a decrease of nearly equal amount; that the relation of height to duration of spring floods has remained nearly uniform throughout the record, while summer and fall floods show a tendency to be relatively shorter and sharper at the two ends of the record than through the middle of the record.

As to the duration of flood stages and amount of flood discharge (see Figs. 7 and 8 for discharge curves), Col. Burr says:

The diagrams for total annual days of flood and for days of flood per inch of precipitation are approximately parallel to the precipitation diagram, indicating quite clearly that, when eccentricities are eliminated by averages over a period of 10 years or more, the duration of floods is dependent largely upon the amount of precipitation and that the duration of floods per inch of precipitation increases with the latter. The same characteristics are recognizable in regard to annual discharge, annual flood discharge, the percentage of annual discharge that runs off during flood stages, and the flood discharge per inch of precipitation. So marked are these characteristics that it is difficult if not impracticable to eliminate the influence of precipitation and to ascertain the existence and effect of any minor influence such as variable forest conditions.

In an endeavor to reach a conclusion in this matter attention may best be directed to the five diagrams that are exceptionally similar in outline and that show the number of days of flood, the days of flood per inch of precipitation, the annual flood discharge, the ratio of flood discharge to total discharge, and the flood discharge per inch of precipitation. These diagrams show a general tendency toward a decrease in duration and run-off for floods to about 1880 and an increase since that time. In this respect they follow in a general way the diagram for precipitation, but differences worth noting are observable in the diagrams showing the number of flood days per inch of precipitation and the ratio of flood discharge to total discharge, in which diagrams the influence of variations in precipitation is eliminated as far as possible. It will be noted that at the end of the record the precipitation was nearly the same as in 1880, being about 2.5% greater, while the number of flood days and the ratio of flood discharge to total discharge were both about 20% larger than in 1880. It may be held that this record is not sufficiently long and definite to show that the material increase in forested areas since 1880 is responsible for this apparent increase in flood duration and discharge but the indications point that way. In any case it may be stated that nothing in the record indicates that deforestation has been accompanied by an increase or reforestation by a decrease in flood duration and discharge.

Deforestation has not increased the duration of flood stages nor the amount of run-off or discharge during flood stages. There are indications that flood duration and discharge have increased since 1880, coincident with a material increase in forested areas, but a longer period of observation is necessary to a definite conclusion.

Figs. 9 and 10 present curves for duration and height of low-water stages. We reprint a portion of Col. Burr's comment on these diagrams:

The diagrams on Fig. 10 (10-year periods) again illustrate the fallibility of relatively short records, since the portions of them subsequent to 1864 and covering a period of more than 40 years would be misleading if considered apart from the record prior to that date. The data indicate that in the later sixties and early seventies the low-water stage of the river averaged higher and of less duration than it did before and after that time. In diagrams 4 and 5 an attempt has been made to eliminate the effect of variations in precipitation, and the results are essentially similar to those apparent in the preceding diagrams. It is also worthy of note that the indications as to progressive changes in low-water flow are in a general way complementary to the tendencies apparent in the data regarding flood stages, with some difference as to time of maximum effect, and to this extent they support each other. In each case there are no indications that any detrimental effect has resulted from deforestation or that any beneficial effect has followed reforestation and such indications as are apparent point the other way.

In connection with this part of the subject, a close analysis has been made by Mr. Stevens of the varying conditions of storage in Winnepesaukee Lake and of the withdrawal of water from the Concord and Nashua valleys for the supply of Boston and its Metropolitan District. His conclusions as to lake storage are that it is in no way responsible for any progressive low-water changes. The effect of water withdrawal for the supply of Boston is not easily ascertainable, with any exactness. The Boston and Metropolitan water-supply from this basin is dependent on the storage of surface water. Since 1875 it has included the run-off of 94.07 sq. mi. and since 1898 the run-off of 118.23 sq. mi. additional, a total since 1898 of 212.30 sq. mi. These areas are respectively 2, 2½ and 4½% of the entire basin. Assuming that none of the low-water flow from these areas has been wasted and that all of it has been withheld from the river, it can at most amount to no more than 4½% of the low-water flow at Lawrence, and may have been less. With a discharge of 2,000 cu. ft. per sec. for a gage height of 4.5 ft., 4½% of the flow amounts to about 90 cu. ft. per sec., and its loss would reduce the gage reading only 0.1-ft. To neither of these causes can the loss of about 0.6-ft. in the average low-water level since 1865-1874 be attributed and this loss, like the gain prior to that decade, when no permanent withdrawals of water were being made, must be charged to other causes. Regarding the gain last mentioned, it is noticeable that it occurred during a period when precipitation, as recorded at Lowell, was normal and uniform throughout.

Bearing in mind that deforestation attained its maximum extent about 1870, it is evident that the data available do not show any ill effects on low-water stages from the loss of forests or any beneficial effects from reforestation amounting to 25% of the entire basin. Such tendencies as are apparent from the data for 60 years seem to indicate contrary effects and a most conservative conclusion can go no further than to state that these tendencies are perhaps not sufficient in magnitude or as to time to warrant a definite opinion that forests affect low-water conditions adversely.

A few sentences from Mr. Stevens' report may be given here:

Curves 2 and 3 (Fig. 10), showing mean minimum gage height and mean number of days per year that stage was below 5, afford an excellent illustration of the fact that studies of precipitation and run-off, covering short periods, are apt to be misleading. Had this study commenced with the year 1865 it would have shown a continuous falling off in the height of low stages and a continuous increase in their duration. Following the curves back, however, to 1850 we find that low-water conditions at that time were, if anything, worse than they have been at any time since.

The report includes eleven large folding sheets, showing "hydrographs of the river for some typical flood and low-water years, including the great floods of 1852, 1895 and 1906." On these sheets, daily temperature and precipitation records for some station as nearly as possible to the headwaters of the upper river are given. Snow depths at Plymouth, in the northern portion of the drainage area, as measured in the open at intervals during the winter," are given for some of the later years of the study. Col. Burr expresses regret that the snow record is limited to recent years and that no records for snow depths in forests and in the mountains are available. He also states that no attempt has been made to analyze the diagrams and that they seem to teach nothing regarding "the normal river conditions other than such as are matters of common knowledge."

General Summary.

Our extracts from Col. Burr's report have already given a number of his conclusions, but for convenience we include here his "summary," even at the risk of slight repetition:

The facts and conclusions that are stated throughout the above discussion, as the various headings have been considered in detail, may be summarized as follows for the basin of the Merrimac River:

Deforestation of the basin continued progressively from the early settlements until about 1800-1870, and since that period forested areas have increased through natural causes by 25% or more of the entire basin, notwithstanding the continuance of lumbering operations.

There has been no decrease in precipitation in the basin as a result of deforestation or any increase with the reforestation of 25% or more of its area. The precipitation for 50 to 90 years at points within the basin or within a few miles of its borders shows tendencies or

cycles that bear no relation to the changes in forest areas.

The average run-off through the river varies with the precipitation over its basin, and the percentage of run-off to precipitation is not appreciably affected by forest changes as great as 25% or more of the basin.

The frequency of floods has not been decreased by reforestation nor increased by deforestation.

Exceptionally high floods have occurred at intervals without respect to forest conditions. Flood heights have not been decreased by forestation or increased by deforestation, and the principal characteristics of floods are unaffected by forest changes. The duration of flood stages and the amount of run-off during such stages have not been affected adversely by deforestation or beneficially by reforestation.

Deforestation has not lessened the height of the river at low water or increased the duration of low-water periods, and the reforestation of 25% or more of the basin has not had any beneficial effect on low stages of the river.

Variations in stream flow are determined essentially by variations in climatic conditions which move in irregular cycles independent of forest changes.

Correct deductions as to climatic variations and as to varying conditions of stream flow may be expected only from the analysis of satisfactory records covering periods of 60 years or more, and conclusions drawn from records extending through 40 years or less may and probably will be misleading or incorrect.

These conclusions are unquestionably correct so far as they relate to conditions in the Merrimac basin during the past 60 years, and they are in accord with the opinions of engineers who have become acquainted with the hydraulics of the river by professional connection therewith through many years of this period. They result alone from an analytical examination of the facts as to precipitation and river stages that have been brought together for the purposes of this study but that come from various independent sources. No consideration has been given to any of the various theories and contentions advanced in the absence of a sufficient record of facts, to prove that forests do or do not beneficially affect stream flow through their influence on conditions as to soil, rate, and time of run-off and other related elements of the problem.

The facts as to the many-sided peculiarities of stream flow, in relation to precipitation, as recorded by the river itself through its varying stages, are a practical and evident verification of theories as to the effects of forests and other minor influences, and these theories as a whole must stand or fall by the results. The facts brought out by this study covering a period of 60 years do not show that reforestation to the extent of 25% of the Merrimac basin has exerted any beneficial effect upon the flow of the river or that the later part of the period of deforestation had any adverse effect thereon. On the contrary there are a number of indications that this reforestation has been coincident with adverse effects on the regimen of the river in some of its aspects. The extent of these effects and the period of years included in them are not, however, sufficient to warrant a definite conclusion at this time that reforestation has been detrimental to the flow of the Merrimac River, and they may be either confirmed or obliterated by the records of the future.

An Engineering Study of the Rapid Transit Situation in Philadelphia.

The Pennsylvania State Railroad Commission has recently published the report made by Ford, Bacon & Davis, of New York City, in the matter of complaints against the Philadelphia Rapid Transit Co. In May, 1910, this firm was commissioned to study the service and equipment in Philadelphia to determine whether certain complaints which had been made were well founded or not.

The examination was first approached by obtaining from the Philadelphia Rapid Transit Co. comprehensive statistics showing the amount and character of the physical property and covering a record of its operations for several years. Similar data were secured, from supervisory commissions and street-railway companies, for New York, Brooklyn, Boston and Chicago.

From the middle of July, 1910, to February, 1911, systematic observations were made of traffic and car service in Philadelphia. Detailed examinations of the car equipment itself in Philadelphia and in the other cities mentioned were also made.

The report at hand comprises a series of tabulations, maps and diagrams, together with explanatory text and recommendations for improvement.

The firm reports that the property and service of the Philadelphia Rapid Transit Co. compare favorably with other large systems, except in two essential features upon which, more largely than on any other, depends the comfort and convenience of the public. The track construction is substantial; the actual track mileage is larger than the average for the population served; the overhead work is of first class design; the power plants are of adequate capacity and of fair economy; and the barn and shop facilities are ample.

However, the car equipment itself is decidedly inferior to that in the other large cities noted. Moreover, the proportion of rush-hour car service to midday service is also considerably less than standard practice, although the average of seats provided per passenger and average total car mileage for the entire day are greater than in the average of other cities. The decreased ratio mentioned shows unreasonable overcrowding at the times of heaviest traffic and lack of economy at other times. The recommendations found in the report therefore relate largely to improvements of the rush-hour car service and car equipment.

In addition, it has been found that the present routing of cars is poorly arranged, from the standards of both public convenience and economical operation. The delivery district trackage is not accessible to lines from all parts of the city. The tracks through the residence districts are not uniformly spaced and, on account of the original competitive conditions, many lines exist on narrow streets with numerous curves and frequent crossings. Messrs. Ford, Bacon & Davis give, in their report, a statement of the general principles which should be followed in this case, and have developed a tentative plan of rerouting the lines of the entire city.

RUSH-HOUR SERVICE.—The traction company was found to schedule at present in winter about 39% more cars during the evening rush hour than on the base schedule in force during the middle of the day. In summer the excess is about 45%. In the other cities noted, as great an increase as 100% over the base schedule is operated. This comparison is cited to prove the feasibility of doubling the service in Philadelphia at times.

It was found that the Philadelphia prepayment cars had a capacity of 38 seated and 38 standing; the cross-seat cars had a capacity of 40 seated and 29 standing, showing that the comfortable standing capacity in general was equal to from 60 to 90% of the seating capacity. This loading was cited as a reasonable limit to be used in computing recommended car movements.

It was recommended that the company furnish sufficient rush-hour service to carry the traffic with the loading mentioned, and to prevent periodical overcrowding and bunching of cars. It was recommended also that platform gates and "full-car" signs should be used to increase the speed and regularity of operation.

The report shows, taking the traffic figures of Oct. 5, 1910, as representing an average loading, that 25% more cars would be required to carry the traffic from maximum points of loading on each line in the busiest half hour of the day. This would result in a 14% addition to the number of large cars in operation. A total of 2,265 cars would be in operation, making necessary the purchase of 489. If these cars were operated at the rush hours, there would be 89% more cars in service then than in the middle of the day, which is considered within the limits of American practice. This additional service is estimated to result in additional car mileage of 11% and an increase of approximately 9% in operating expenses. It is believed that more efficient scheduling in the non-rush hours and the replacement of many small cars by the new larger cars, together with the savings of a comprehensive re-routing scheme, would largely offset the additional expense of rush-hour operation. It is expected that these changes will lead not only to better service for the public, but to a considerable increase in the gross earnings of the company.

CAR EQUIPMENT.—Two-thirds of the 3,292 cars owned by the company were found to be of the original, small, single-truck type, dating back to the original electrification. Of the larger double-truck cars, 272 were made by simply splicing in a center section. As a result 70% of the equipment is unsuited to present conditions. There were found to be 772 large double-truck cars; of these 90% were made into "prepayment" types, by ripping out the cross seats and putting in longitudinal seats of very poor design.

It was recommended that the unsatisfactory prepayment cars be changed by enlarging the platforms, restoring eight cross seats, using higher-backed and more comfortable longitudinal seats, providing a simple emergency release for the air-operated doors, installing straps, rails or stanchions, doubling the number of heaters and lamps, providing suitable fenders and wheel guards, etc.

The following description of the new standard car proposed is given in the report:

NEW STANDARD SURFACE CAR.—It is assumed that the new standard car for Philadelphia will be a two-motor, prepayment car with windows capable of large opening, which would make it suitable for both summer and winter operation. If the body is 28 ft. long with platforms 5 ft. 6 ins. long (inside), mounted on reversed, center-bearing, maximum-traction trucks, the overhang of car without projecting fender when operated on the standard track curve at 50-ft. street intersections would be approximately the same as the bumper on the present "pay-within" car, and about 18 ins. less than the overhang of the fender on the present car. Consequently the length of body might be 30 ft. if found desirable, or even longer if operated on some lines. For the purposes of this report we have assumed the use of a 28-ft. cross-seat car as generally practicable, with seating and standing capacity of 76 passengers on the basis recommended.

This, for a car 8 ft. 4 ins. wide with skeleton sides and 34-in. seats, would permit of an aisle 28 ins. wide. With exits from both rear and front platforms, and with the recommended limit of car load, this width of aisle should be found practicable from an operating standpoint.

Our recommendations upon the points of design especially affecting the traveling public, are as follows:

The height of the step should be from 14 to 15 ins. and the width of step at least 11 ins. The step should be folding or protected and should have safety tread.

The folding platform doors should be mechanically operated by motorman or conductor and should be kept closed while the car is in motion. The bulkheads should be open and all passageways should be at least 23 ins. in the clear. The conductor should be located on the rear platform behind a suitable rail or steadying device.

The seats and backs should be of spring rattan, of comfortable height and properly sloped.

The vestibule windows should be given full opening. Monitors should be fixed shut and there should be provided with sufficient roof, or monitor, automatic ventilators.

There should be installed 16 electric heaters of a total rated capacity of 12 amps. Twenty 16-cp. incandescent lamps should be used; 15 inside the car arranged in single sockets on the ceiling and lower decks, one on each platform, two for destination signs and one for headlight.

There should be a passengers' push-button signal on each side post of car with electric bell or buzzer on platforms.

Mechanical sanders and automatic wheel guards should be installed. Draw-heads should not project beyond bumpers.

The Locomotive Works of the London & Northwestern Ry. at Crewe, England, have just turned out their 5,000th engine, which is appropriately named "Coronation." It is of the 4-4-0 type, with coupled wheels 6 ft. 9 ins. in diameter. The cylinders, which are placed inside the frames, have a bore of 20½ ins. and a stroke of 26 ins., and the boiler, 5 ft. 2 ins. in diameter and 11 ft. 9¾ ins. long, is designed for a working pressure of 175 lbs. per sq. in. The total heating surface is 1,849.65 sq. ft., of which 1,004.96 sq. ft. are accounted for by 168 tubes of 1¾ ins. external diameter, 380.44 sq. ft. by 24 tubes, 5 ins. in diameter, 161.75 sq. ft. by the fire box and 302.5 sq. ft. by 24 sets of superheater tubes. The total weight of the engine in working order is 59 tons 17 cwt., or 99 tons 2 cwt., inclusive of the tender, which has a weight of 39 tons 5 cwt., and holds 3,000 gallons of water and seven tons of coal.

By way of contrast it may be mentioned that the first engines made at Crewe were of the 2-4-0 type, weighing about 19½ tons, and with outside cylinders measuring 15 ins. by 20 ins. Those intended for passenger work had coupled wheels 5 ft. 6 ins. or 6 ft. in diameter, while the goods engines had 5-ft. wheels. The Crewe works were established by the Grand Junction Ry. in 1843, three years before it was combined with the London & Birmingham and the Manchester & Birmingham companies to form the London & Northwestern.—London "Times," June 28.

Novel Methods in the Brooklyn Fourth Ave. Subway Construction, New York City: Section 11-A-3.

By S. P. BROWN,* M. Am. Soc. C. E.

The portion of the so-called Fourth Avenue Rapid Transit Subway in Brooklyn which is now under construction extends from the Brooklyn end of the Manhattan Bridge along the Flatbush Avenue extension, Fulton St., Ashland Place and Fourth Ave. to 43d St. Eventually the subway will be extended, one branch continuing along Fourth Ave. to Fort Hamilton and another branch turning off Fourth Ave. at 40th St. to Coney Island. The general type of structure used is shown by the cross-section Fig. 4 herewith.

The first section, extending from the Manhattan Bridge to Willoughby St., is being constructed by Smith, Scott & Co., of St. Louis, Mo. In this section there were neither surface nor subsurface structures of any consequence to contend with, except at cross streets, so that the excavation is being made largely by scraper bucket.

The next two sections, extending to Sackett St. on Fourth Ave., are being constructed by William Bradley, of New York. Here cableways are principally used for handling the excavation, which is loaded into buckets by hand and hauled away in trucks. Gravity concrete mixers are used and the concrete is distributed by teams as desired along the street. Here the reinforced concrete section of the subway has largely been changed to a structural steel section, at the instance of the contractor. These sections, being in deep cuts through wet ground and with many variations in line and grade, form a difficult piece of construction.

The fourth and sixth sections (11-A-2 and 11-A-4) extending from Sackett St. to between 9th and 10th Sts. and from between 27th and 28th Sts. to 43d St., are being built by the E. E. Smith Contracting Co., of Philadelphia. Here movable derricks are used which dump buckets, loaded by hand, into trucks on the surface. The concrete is mixed by several movable mixers of various types, from which the concrete is discharged down chutes from the street surface to the forms below.

The fifth section (11-A-3), extending along Fourth Ave., between 10th and 27th Sts., which is being built by the Tide-Water Building Co. and Thomas B. Bryson, of New York, is the section particularly considered in this article. Section 11-A-3 is largely in high ground, consisting of loam, sand, gravel and boulders. The avenue is 120 ft. wide between building lines, with a 20-ft. parkway in the center and 25-ft. sidewalks. It runs along the side hill between Prospect Park and the Gowanus Canal, cutting cross-streets that have a very steep grade and a correspondingly heavy runoff in times of severe rains.

*Chief Engineer, Tide-Water Bldg. Co. & Thos B. Bryson; 641 Fourth Ave., Brooklyn, New York City.

Subsurface Structures.

SEWER DIVERSION.—After a study of the conditions, it was decided that the first thing to be done was to rebuild the sewers, providing special catchbasins to intercept the water on the up-hill side of the subway cut. Large interceptor sewers were therefore built in the east sidewalk, with depressed crossings under the subway at 12th and 26th Sts. This not only took care of the floods which otherwise would have washed into the cut, but also cut out all sewers crossing Fourth Ave. above the excavation grade of the subway. The small west-side sewers were built after the main excavation had commenced. The off-line sewers were sublet to the Donegan-Redmond Co., of Brooklyn.

The most interesting sewer work was that on Third Ave. (a block west of the subway), between 26th and 27th Sts. Here the old sewer had to be enlarged to accommodate the flow from the southern interceptor sewer on Fourth Ave., which was to discharge into Third Ave. at 26th St. instead of at 27th St. as formerly. The principal difficulties encountered here were the heavy vehicular traffic, a double-track trolley line, and the usual subsurface structures, as well as a normal dry-weather flow that nearly half filled the old sewer; all of which had to be maintained.

It was finally decided to build the new sewer around the old one, using the old invert, which was found to be in excellent condition, as part of the invert of the new sewer; see Fig. 1. With this in view, the old sewer was uncovered and sheeting was driven 2 ft. outside the old walls. The old footings were broken off in a reversed corbel and the new footings were bonded into them and spread out a foot under the toe of the sheeting. New concrete walls, 2 ft. thick, were carried up on these footings to the roof grade of the new sewer. Then the old sewer was broken out. A longitudinal cofferdam being now built down the center of the old invert, the flow was diverted first from one side and then from the other, and the bottom where the old walls were broken down was pointed off to give a smooth invert throughout. After this, the placing of the roof was a simple matter.

By this method, in spite of the large flow to be maintained, the only opening in the street was a trench of the actual width of the new sewer. As a cableway was used, the contractor's trucks did not encumber the narrow passage, and comparatively little inconvenience was caused to the public. One side of Third Ave. was given over to the street-car company and the other was devoted to vehicular traffic.

In the case of the Fourth Ave. sewers, the sheeting on one side of the sewer trench was left in place to form the sheeting for the pipe galleries which were to be built adjacent to the subway as part of the contract; the sheeting also served to help support the temporary roadway and to aid in excavating for the changes in water pipes, ducts, etc.

PIPES AND DUCTS.—A special contract was made with the Water Department by which the firm was to install certain new hydrants, gates and connections furnished by the city, in return for which all water pipes crossing Fourth Ave. were discontinued during construction and the city was to loan the firm 1,400 lin. ft. of each size of water pipe then running along the avenue. All longitudinal water pipes were, therefore, taken up and relaid in the sidewalks, in the permanent positions they were to occupy in the pipe galleries, which were to be built around them later. Since then, however, the pipe galleries have been abandoned entirely, though not until this pipe work had been completed.

All but three gas pipes crossing the avenue were also discontinued, and steel pipes were temporarily laid in the sidewalks to replace the longitudinal pipes. Where the three high-pressure mains crossing Fourth Ave. had to be maintained, heavy masts were erected on each side of the avenue and steel pipes suspended about 20 ft. above the street grade by steel cables anchored back in the cross-streets.

The electric underground cables were transferred to permanent ducts in the west sidewalk after being temporarily suspended from the fence along the side of the cut. These cables carried the light and power for Coney Island at 6,600 volts. Telephone, telegraph, fire-alarm and electric lighting wires on overhead circuits were either permanently removed from Fourth Ave. or carried temporarily on poles on the fence line of the cut. Cross cables were suspended overhead.

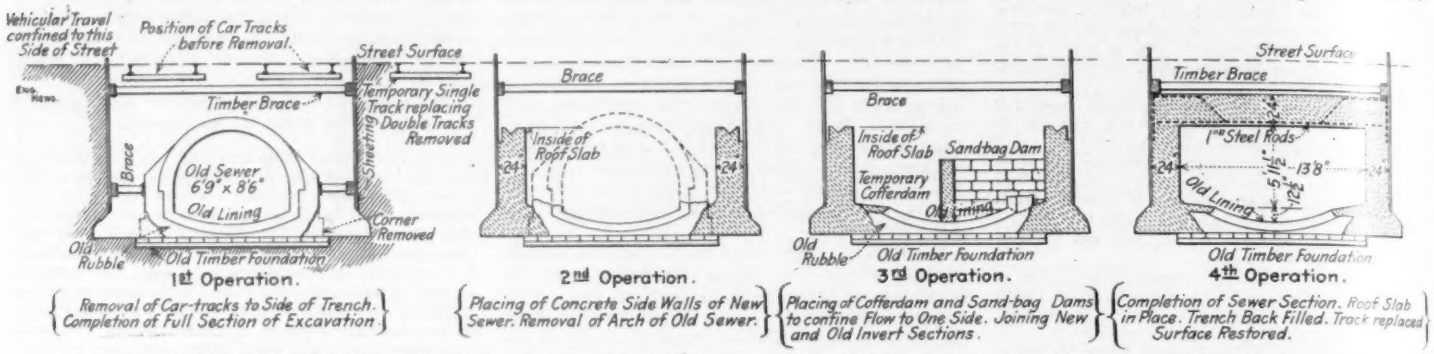


FIG. 1. RECONSTRUCTION OF THIRD AVE. SEWER NEAR 26TH ST., IN CONNECTION WITH FOURTH AVE. SUBWAY CONSTRUCTION.

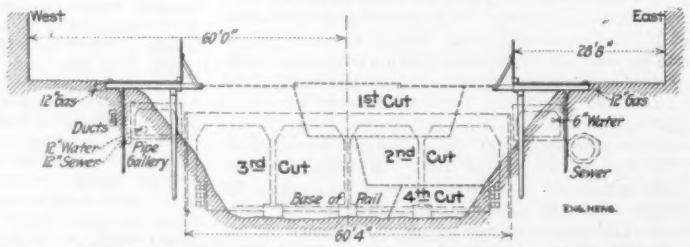


FIG. 2. DIAGRAM SHOWING SEQUENCE OF STEAM-SHOVEL CUTS, TIDE-WATER—BRYSON SECTION OF FOURTH AVE. SUBWAY.

Roadway.

As it was necessary to maintain one-half the thoroughfare for traffic at all times, and at the same time provides access to all property, a temporary wooden roadway 12½ ft. wide was con-

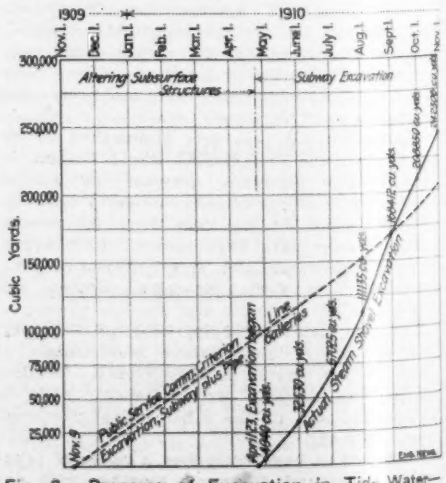


Fig. 3. Progress of Excavation in Tide-Water—Bryson Contract: Expected and Actual Excavation.

structed on each side of the avenue. This was done while the subsurface structures were being altered.

Along each side of the avenue was driven a line of wooden piles spaced 5 to 7 ft. apart and about 30 ins. out from the curb, the piles coming just outside the neat line of the subway. Alside of the roadway, leaving a 12½-ft. drive, and an 8-ft. board fence was built continuously around each block. At the cross streets this fence was left 2 ft. lower than elsewhere, the top being pieced out with chicken wire, so that teams approaching a bridge could avoid collisions. The view Fig. 9 shows the appearance a very steep slope to the subgrade of the subway proper. Thus no sheeting was required in the subway cut until after the steam shovel was finished, the only timbering used prior to this time being for bridges at the cross streets and the 10 × 10-in. rakers along the sides of the cut used to reinforce the temporary roadway.

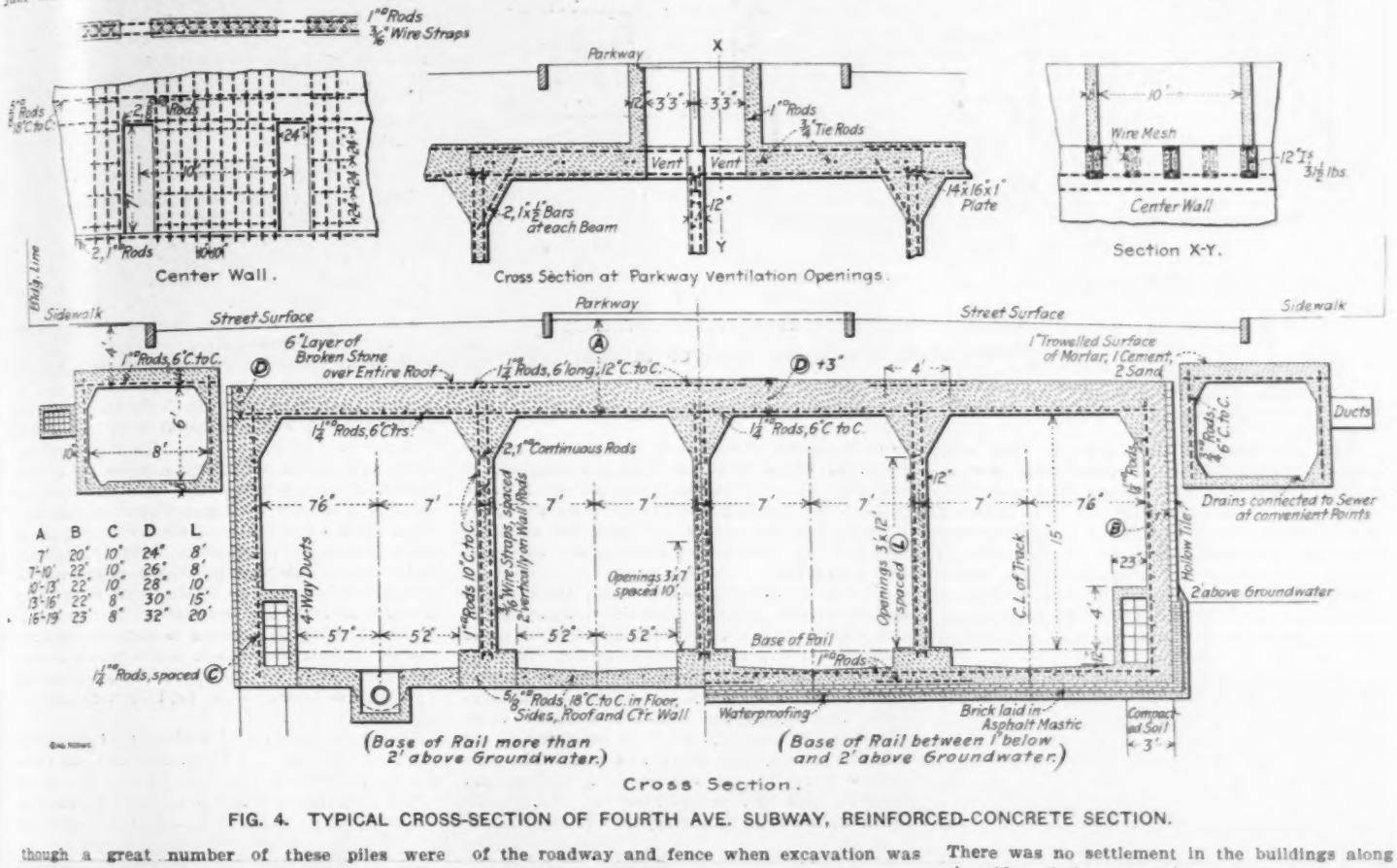


FIG. 4. TYPICAL CROSS-SECTION OF FOURTH AVE. SUBWAY, REINFORCED-CONCRETE SECTION.

though a great number of these piles were broken and broomed in driving (as described in Engineering News of Feb. 23, 1911), they answered their purpose satisfactorily, supporting the temporary roadway without an accident until the steam shovel had passed and 10 × 10-in. rakers could be placed which acted both as a support and restrained any tendency of the roadway to move toward the cut. The piles were capped with 12 × 12-in. long-leaf yellow pine timber, which supported the outer ends of 3 × 10-in. floor joists 14 ft. long, spaced 12 ins. c. to c. The rear end of these joists rested on a 6 × 8-in. stringer which was fastened to the sewer sheeting.

The roadway floor consists of one layer of 3-in. yellow pine covered with a wearing surface of 3-in. beech and maple. Experience has proved that beech and maple wear better than oak, hickory, pine or spruce, and the cost is very little greater than the latter when bought in quantities. Guard timbers were placed on each of the roadway and fence when excavation was practically complete.

All of these alterations were preliminary to the main subway job, the excavation for which was not started until nearly the first of May, 1910. With this preliminary work done, however, the avenue was left entirely free from subsurface obstructions, thus allowing the use of a steam shovel.

Excavation and Invert Concreting.

STEAM-SHOVEL EXCAVATION.—A Model 60 (70-ton) Marion steam shovel with a 2½-cu. yd. dipper was used on the main excavation and was the principal feature on this portion of the work. The excavation taken out by the steam shovel was made in four cuts, as shown in Fig. 2, which totalled about 85% of the subway and 50% of the pipe-gallery excavation. The banks sloped from near the top of the sewer sheeting to a point at the subgrade of the pipe galleries on the outside line of the subway and thence at

There was no settlement in the buildings along the sides of the cut, and, as all work was confined inside the fences around the job, there was the least possible inconvenience to the public. Figs. 7 to 9 illustrate the shovel work.

The excavation was handled from the steam shovel in 4-yd. side-dump Continental cars hauled by 20-ton 36-in. gage American locomotives. These trains were dumped on scows from an elevated tipple on the firm's dock at 19th St. and Gowanus Canal, the track being laid from the job on Fourth Ave. directly down 19th St. to the water. This tipple was built with an overhang (see view Fig. 11), so that the trains stood 10 ft. outside the stringpiece of the dock when dumping. An apron threw the material into the center of the boat, thus avoiding the necessity of trimming scows. While a part of this spoil was used in reclamation, the bulk was dumped at sea, all spoil being handled by the Moran Towing & Transportation Co., of New York.

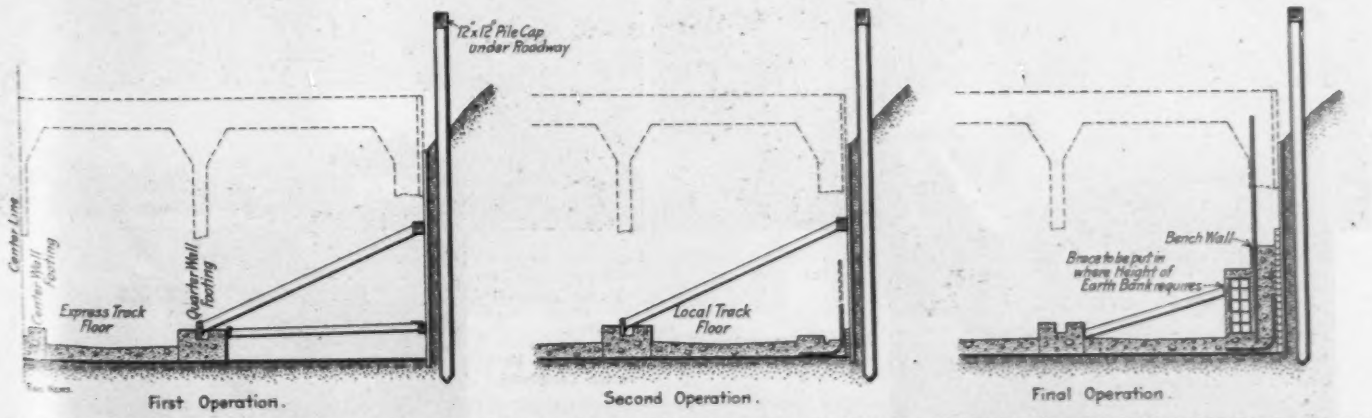


FIG. 5. ARRANGEMENT OF SHEETING FOR PLACING CONCRETE OF SIDE-WALL FOOTINGS; TIDE-WATER—BRYSON SECTION OF FOURTH AVE. SUBWAY.

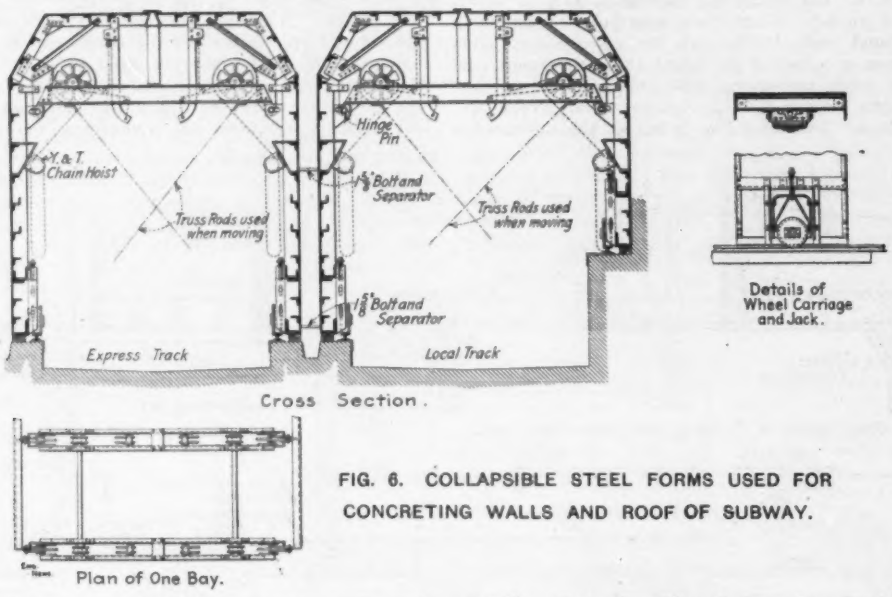


FIG. 6. COLLAPSIBLE STEEL FORMS USED FOR CONCRETING WALLS AND ROOF OF SUBWAY.

With this method of excavation very rapid progress was attained, at a low cost. For instance, during July and August, 1910, over 100,000 cu. yds. (place measurement) was excavated, which amount the Public Service Commission's engineers had estimated would require about six months by former subway methods. The shovel was started about May 1, 1910, and finished its work about Dec. 1, 1910. The excavation progress diagram, Fig. 3, is interesting as showing the high rate of progress attained.

SHEETING AND BOTTOM CONCRETE.—After the shovel had finished its fourth cut, bringing the excavation to grade in all but the outside corners, the concrete footings from the center wall, the two quarter (intermediate) walls and the floor slabs of the two express (center) tracks were put in. Brown-Hoist locomotive cranes were then used to trim up the final excavation along the sides of the cut and set the sheeting, which extended from the subgrade of the pipe galleries to the subgrade of the subway. This sheeting was braced from the quarter-wall footings to two rangers, one about 18 ins. and the other about 10 ft. above subgrade. (See Fig. 5.)

As fast as the sheeting was set, the concrete for the outside or local track slabs, duct benches and outside wall footings was placed and the lower rangers and braces were removed (Fig. 10). The next operations were: placing hollow tile against the sheeting, setting up the side-wall reinforcement, and laying the electrical conduits (consisting of 10 four-way ducts on each side of the tunnel). Finally the bench and side walls were concreted up to the top sheeting ranger and these rangers and braces removed, making the bench walls act as retaining walls to support the sheeting and leaving no timber in the cut except the bridges at cross streets and the 10 10-in. rakers under the side roadways. Where the bank was unusually high, extra heavy sheeting was used and a supplementary brace placed between the quarter-wall footing and the bench wall.

The bottom concrete is placed by means of specially designed side-dump cars holding 2 cu. yds., so arranged that they shoot the concrete directly into the footing forms and track slabs. For the bench walls 1-yd. Lockwood automatic buckets are handled by a crane and wooden panel forms are used.

Wall and Roof Concreting.

The main concrete work consists of building the center wall, quarter walls, outside walls (above the bench walls), and the roof. This is all done in one operation by the use of large traveling steel forms, patented and built by the Blaw Collapsible Steel Centering Co., of Pittsburg, Pa., who also collaborated with the contractor in their design.

STEEL FORMS.—The steel traveling forms (Fig. 6) are built in short sections bolted together, each section being 5 ft. long and complete in itself, weighing about 8,000 lbs., or 1,600 lbs. per foot of single tube. Eight of these sections are bolted together to make up a 40-ft. length of form, which is the standard length now in use, although at times 80-ft. sections have been used. While the forms for each tube are built and moved separately, when set up ready for concrete all four tubes are bolted together every five feet by two 1⅝-in. bolts 17 ins. long, inclosed in 2-in. pipe separators. In this way no timber bracing is required to support the forms except against the outside walls, where there is nothing to bolt to, of course. This permits the two express tracks to be kept open to traffic at all times.

The forms are built of 3/16-in. mild steel plates stiffened by 8-in. and 10-in. channels and heavy angles and Z-bars (see Fig. 6). The top panels, which form the roof and haunches, are hinged to the 10-in. cross channels about 18 ins. below the

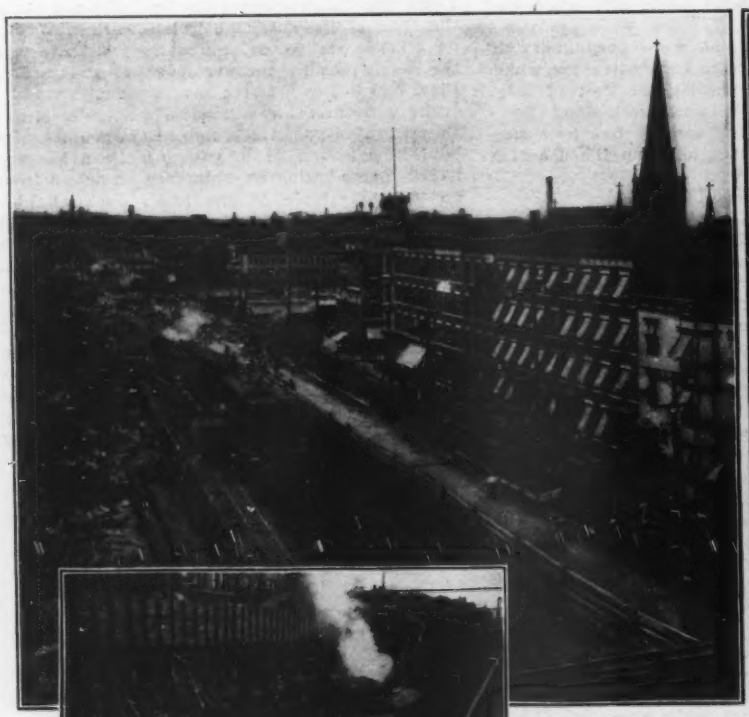


Fig. 7. Steam-Shovel Making First Cut.

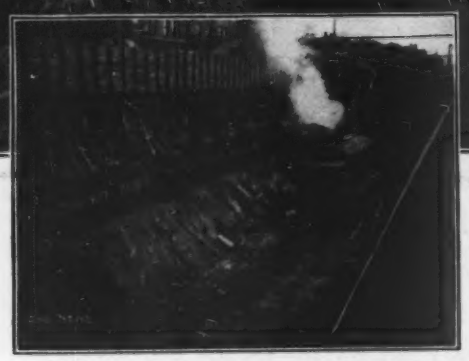


Fig. 8. Steam-Shovel in Third Cut.

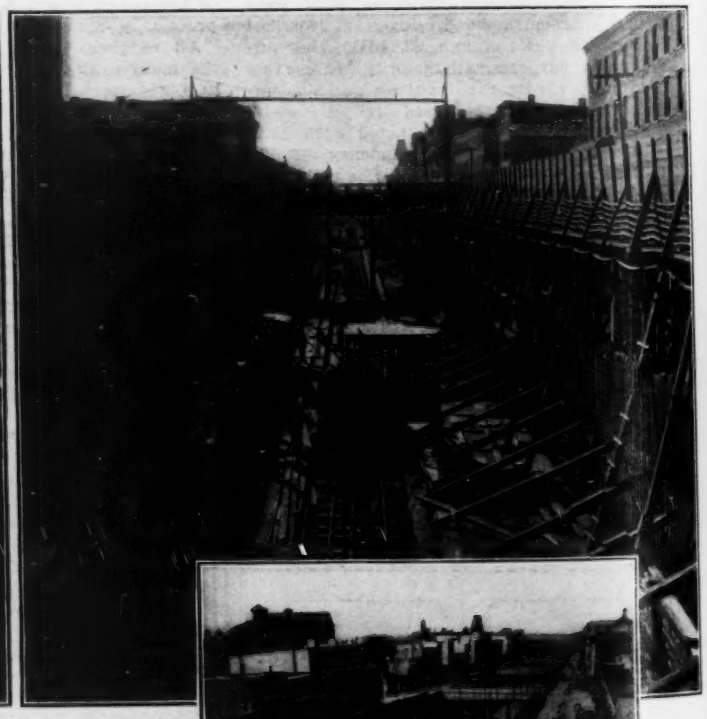


Fig. 10. Side-Wall Footing and Duct Work. (Unsheeted in Right Background).

Fig. 9. Shovel Excavation Completed.

FIGS. 7-10. EXCAVATION AND FOOTING-CONCRETE OPERATIONS ON TIDE-WATER—BRYSON SECTION OF FOURTH AVE. SUBWAY.

FIG. 11. DUMPING FROM TRESTLE CANTILEVERED OVER DOCK TO LOAD SCOWS CENTRALLY.

spring of the haunch and close enough to the walls to insure a clear draft at the upper outside corners when slacking. The hinge pins are supported by slotted gusset plates riveted to the vertical rib channels of the side panels. These vertical channels are also linked to the hinge plates about 12 ins. above the hinge pins so that, when the roof panels are slacked down, the tops of the side panels are drawn in 3 ins. automatically. Conversely, when the roof panels are raised to their proper position, the tops of the side panels are automatically set to their proper line and dimension. Thus a single strut between the cross channels and the roof panels absolutely fixes and braces the entire upper portion of the forms. An opening about 3 ins. wide is left down the center of each tube between the roof panels to allow clearance in slacking, and this is covered by a steel plate 6 × ⅛-in. × 5 ft. long which is held in position by a little wooden block tacked to each end and dropped into the 3-in. slot.

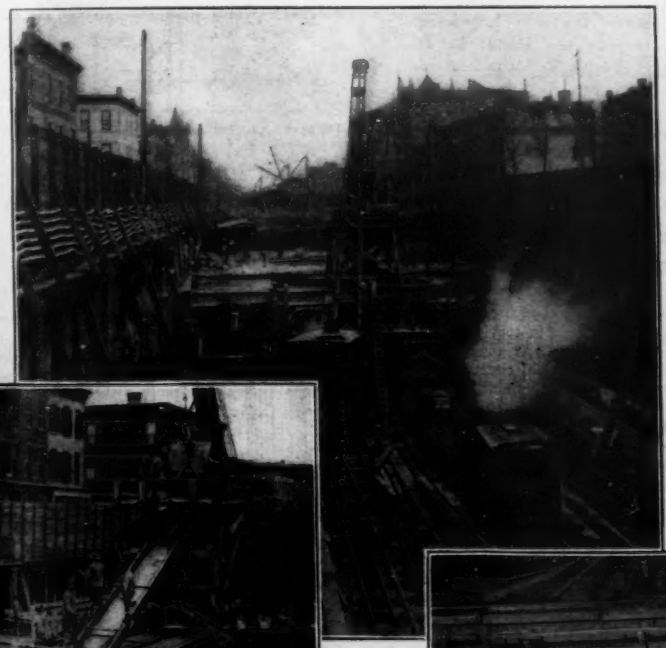
The roof panels are raised and lowered by Yale & Towne chain hoists, four to each 40 ft. single tube form. These chain hoists, by means of a large sheave wheel, turn a longitudinal shaft, on each side of the tube, supported by the cross channels and running the entire length of the form. Between each pair of cross channels is a spool on which two ⅜-in. plow steel cables are wound. These cables, running over idlers, are attached to swinging arms by which the roof panels are raised and lowered.

In the side panels the spacing bolts are placed close to the vertical channels and about 6 ins. and 7 ft., respectively, above the base of the forms. Hand-holes are built into the panels immediately above each bolt hole. At the foot of each 5-ft. section a 14-in. double-flanged wheel is placed in a heavy steel frame, which is given a vertical adjustment by means of a jack rigidly attached to the side panel. Very heavy axles are used in these wheels; they are of such length that the wheel has a lateral play of 3 ins. By this means the bottom edges of the forms are drawn in when the forms are slacked and any necessary lateral adjustment is allowed for in setting. The wheels run on a 60-lb. rail, riveted to a heavy angle which fits over the corner of the wall footings.

After a section of concrete has set, the spacing bolts of the forms are taken out, the forms slacked (Fig. 14), and crossed truss-rods with turnbuckles put in and tightened up diagonally across the tubes to hold the forms from rocking while being moved. A locomotive then draws them ahead, one tube at a time.

The wall reinforcing steel and cross passage forms are all set in advance, so that when the steel forms are moved ahead the remaining reinforcement consists of roof rods or beams only (Fig. 16). With this system, the time lost in moving and setting forms is reduced to a minimum and little skilled labor is required.

PLACING CONCRETE.—The concrete is brought from the mixing plant, which is centrally located, in 2-yd. Lockwood automatic buckets. These buckets are operated by the man on the crane who hoists them, thus greatly increasing the speed of placing. The buckets are dumped into an elevated hopper standing on and



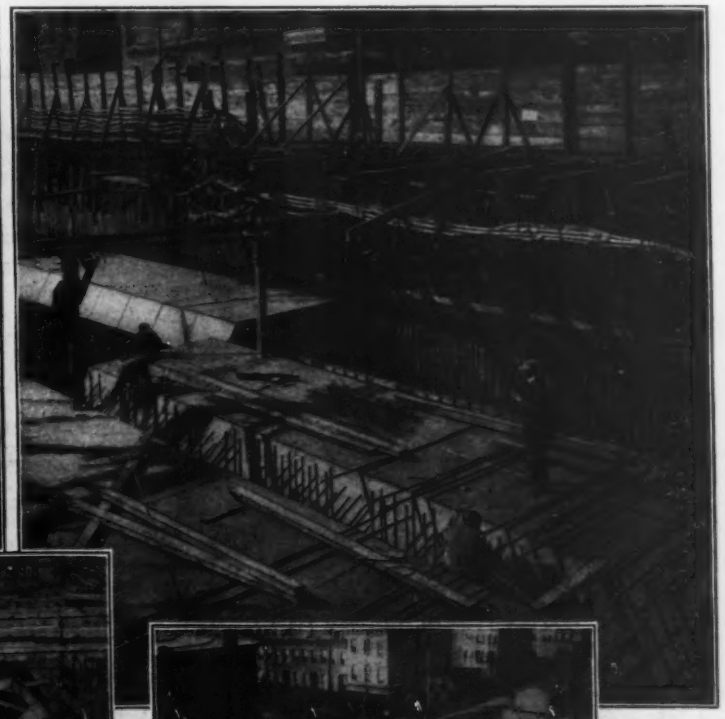


Fig. 12. Concreting in Progress: Locomotive Crane Delivering Concrete Buckets to Hopper.

Fig. 13. Concreting in Progress; Chutes Distributing Concrete on Top of Forms.

Fig. 14. Collapsing a Section of Form to Draw Out of Finished Work.

Fig. 16. Placing Reinforcing Rods over the Forms.

Fig. 15. Two Sections of Form in Second Position.

FIGS 12-16. CONCRETING OPERATIONS WITH COLLAPSIBLE STEEL FORMS; TIDE-WATER—BRYSON SECTION OF FOURTH AVE. SUBWAY, BROOKLYN.

12 ft. above the top of the forms (Fig. 12). The concrete runs by gravity from the hopper down an inclined chute (Fig. 13), which in the case of a 40-ft. section reaches to the finished arch last completed and has trap doors in its bottom through which the concrete can be discharged at any point along its length. The chute is pivoted at the hopper and has a wheel at its far end, so that it can be swung from side to side of the arch at will. Still further flexibility is gained by the use of light metal chutes that convey the concrete from the trapdoors in the main chute to any point of the arch desired. These metal chutes are also used to extend the main chute when an 80-ft. arch is being concreted.

Under this procedure, with a 1-yd. Ransome mixer, a 40-ft. section of complete four-track subway, above the benches, is concreted in about six hours. Such a section contains from 300 to 350 cu. yds. In the case of the last 40-ft. section concreted at this writing, containing 305 cu. yds., 5½ hours elapsed from the time that the mixer was started to the time that the last bucket of concrete was placed.

In the stations (at Prospect Ave. and at 25th St.), where structural-steel framing is used instead of steel rod reinforcement, the large steel forms are used with additional collapsible wooden forms for jack arches, side walls and roof extensions over the platforms. Here the locomotive crane stands on top of the steel work and hoists through the beams.

CONCRETE PLANT.—The concrete material, as well as most of the other material for construction, is received at the firm's dock at 19th St. and Gowanus Canal, where a Brown-Hoist locomotive crane handles the sand, gravel, steel and heavy materials. The gravel and sand are unloaded with a clamshell bucket into overhead bins from which 4-yd. side-dump cars are loaded through swinging gates. The cement house at the dock is divided into halves by a loading track, each half accommodating 15,000 bags of cement. Giant and Edison cement are used, each being kept separate to its own half of the cement house.

The concrete-mixing plant is located at the corner of 19th St. and Fourth Ave. The sand and gravel cars come in at the street grade over depressed bins which form the main storage supply for the mixer. The sand bin holds 1,000 cu. yds. and the gravel bin 2,000 cu. yds. Below the material bins two Jeffrey belt conveyors carry the sand and gravel to a belt and bucket elevator which raises the materials to small bins over the mixer, holding 100 cu. yds. of gravel and 50 cu. yds. of sand. From these bins, the sand and gravel run by gravity to the measuring hoppers, where the cement and water are added, and from which the gross aggregate passes through a Ransome 1-yd. mixer into buckets or cars on either of two parallel tracks underneath, at about the subway grade. The cement comes in on the sand and gravel tracks and is unloaded by hand at the mixer, there being storage for about 2,000 bags on the mixer platform.

While in operation, this mixing plant turns out on an average about 50 cu. yds. of 1:2½:4½ concrete an hour, measured in place, a five-bag batch being slightly less than a yard.

HEATING.—In cold weather live steam is used in the large bins to break up the lumps, and coils in the small bins take the frost out of the sand and gravel on their way to the mixer. The greatest good, however, is done by heating the water. The exhaust from the heating coils in the small bins is discharged into a water tank on top of the mixing plant through a perforated pipe, heating the water to about 120° F. This arrangement produces concrete at from 50° to 70° F. in the coldest weather.

When concreting, canvas curtains are hung at each end of each tube from the steel forms, completely stopping any circulation of air. Large salamanders charged with coke are kept burning in these chambers during concreting and for some time afterward, so that the concrete may be kept warm while it is setting. The top is covered with salt hay, and in very severe weather a large canvas tarpaulin is drawn over the whole thing. Under this system concreting is carried on successfully when the outside temperature is down in the neighborhood of zero.

SYSTEMATIZATION.—In the printed forms used for foremen's reports, storehouse requisitions, discharge slips and the like, nothing unusual has been attempted. The storehouse and tool accounts are kept in card form. The pay sheet is regular, with special columns to show the denomination of currency going into each envelope and to be drawn from the bank. Cardboard pay checks are used, signed by the chief timekeeper, dated for each pay and punched by the particular timekeeper who gives them out. A new ticket is issued for each pay, the old check forming a receipt for pay received. The men are numbered according to position held and the class of work upon which they are employed, the key numbers being the same as those used in distributing labor and bills. Daily cost distribution cards for labor on concrete and excavation, and force account cards showing the distribution of labor and rates in each gang, form a useful record, both from day to day and as a permanent file.

CONCLUSION.—The methods employed in the construction of Section 11-A-3, while unusual in city subway construction, would not excite particular comment on railroad work in the country, nor could they have been developed under different physical surroundings. They represent more a logical development governed by local conditions than a radical change in accepted subway traditions. Had cross timbering in the cut been necessary, the entire plan of construction would have been different.

The work is being done for the City of New York under the Public Service Commission for the First District: Alfred Craven, Acting Chief Engineer; Frederick C. Noble, Division Engineer; H. L. Oestreich, Sen. Asst. Division Engineer; Geo. Murgatroid, Engineer in the Field; L. D. Fouquet, Engineer, and Joseph L. Hunt, Asst. Engineer of the Sewer Division.

For the Tide-Water Building Company and T. B. Bryson, Robert Eldredge is General Superintendent, C. T. Wilson is Assistant Engineer and the writer is Chief Engineer.

The Preservation of Trestle Stringers and Caps by Means of Housing.*

By HUNTER McDONALD,† M. Am. Soc. C. E.

In 1893 the writer had charge of the renewal of a number of combination spans which had been in service about 20 years. The main and second braces and top chords were of yellow pine, and since their original construction all had been carefully covered with weatherboarding on the sides and tin on the tops, the latter carefully fastened to sheathing, with air space beneath, and soldered. All of the timber upon removal was found to be sound, and was used again for trestle stringers.

The value of such protection had been previously observed, and in 1887 the plan of protecting intermediate sills on double-deck trestles with sheets of galvanized iron was adopted. The sheets extended about 3 ins. over the sides and 6 ins. over the ends, and the posts of the upper section were held in place by allowing the drift bolts, passing through the cap into the posts below, to extend about 6 ins. above the cap, through the galvanized iron protection and into the feet of the upper section posts. In a number of instances the top caps were similarly protected about the same time.

The intermediate caps were found to be sound in 1906 when the trestles were filled. The top caps, although the housing had not been well maintained and the stringers rested directly on the galvanized iron, with drift bolts through it, did not require renewal until 1908.

About 1900 the writer had occasion to notice a large pile of stringers which had been removed from the trestles on the Cincinnati Southern R. R. These stringers were put in place about 1879, and had been protected with galvanized iron according to the designs of Mr. G. Bouscaren, Chief Engineer. The ties rested directly on the stringers, holes being made at frequent intervals in the galvanized iron in order to confine the ties to position. Col. G. B. Nicholson, Chief Engineer of the Cincinnati Southern R. R., was present with the writer at the time these stringers were examined and told him that the galvanized iron had not been carefully removed as it rusted out, nor had any great effort been made to keep it from creeping or leaking. Trestles which were still covered were examined, and many leaks found in the iron and many sheets found out of position. The condition of the stringers removed showed remarkable preservation, rot occurring only in local spots around bolt holes—and at points where the protection had been improperly maintained. This examination convinced the writer that, whether the galvanized iron was properly maintained or not, the results obtained fully justified its use, and since 1900 stringers and caps when renewed on the lines under his charge have been so protected.

At that time the standard trestle consisted of two 8 x 16-in. stringers on each side, packed 1 in. apart, each pair being spaced 6 ft. apart on the inside. The chords were confined to the caps by what was called "stag bolts"—that is, drift bolts driven slantingly through their sides into the caps. The ties were 8 × 8 ins. x 9 ft. 10 ins., and each was doweled to the chord by ⅞ x 5-in. dowels, two dowels to each tie passing into alternate members of the chord on each side. Holes were bored into the tie and the dowels driven in to a tight fit, the corresponding holes in the stringers being a loose fit. The guard rail was directly over the outside member of the chord. Two bolts to each span on each side passed through guard rail, tie, and stringer.

The galvanized iron was applied to the caps in the manner already described, and that on the stringers by jacking up the ties, the dowels following them, inserting the iron under them and lowering them again. The weight of the ties caused the dowels to perforate the sheet iron and settle snugly to place after a slight blow with a hammer. It was supposed that the dowels and guard-rail bolts would confine the iron permanently to position, but subsequent observation showed that the iron at many points crept, always outward, no matter whether on sharply elevated trestles or not. The creeping was very slow, and slots were torn in the iron by every dowel and bolt. Careful observations were made with a view of determining the cause of this creeping, but so far the exact cause has not been ascertained. No doubt it is due to different causes at different points. There are many trestles on which no creeping occurs, others where isolated sheets are found to creep, and others where nearly all the sheets creep. It is believed to be generally due to imperfect bearing of the ties on the stringers, longitudinal vibration of the structure, and vertical movement of stringers on the caps or caps on the piles. The problem of remedying this, and also the leaking of water through the dowel and bolt holes, has been continuously studied, with the result that the method of housing hereafter described has been adopted and has been found to be almost entirely successful, both as to leaking and creeping.

To dispense with the drift bolts through the stringers into the caps, which are not only destructive to the timber, but cause much expense in adjustment for line and surface, a malleable casting, called a "line-brace casting," has been devised. Its form and method of application are shown on accompanying sketches (Figs. 1 and 2).

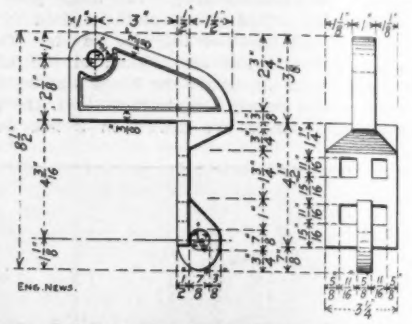


Fig. 1. Detail of Line-Brace Casting for Wooden Trestles Used on Nashville, Chattanooga & St. Louis Ry.

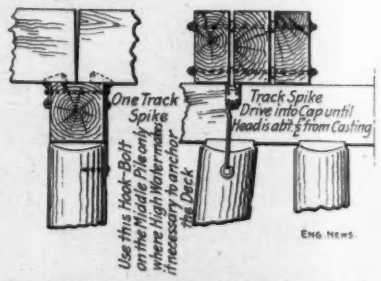

One Track Spike
Track Spike Drive into Cap until Head is abt ½ from Casting
Use this Hook-Bolt on the Middle Pile only where High Water makes it necessary to anchor the Deck
ENG. NEWS.

Fig. 2. Application of Casting Shown in Fig. 1.

When it becomes necessary to line stringers, pull track spikes out of cap then shift stringers to desired position and redrive spikes through most favorable holes.

Clean sand out of holes in castings thoroughly before driving spikes.

*Portion of a paper read before the Engineering Association of the South, Jan. 16, 1911.

†Chief Engineer, Nashville, Chattanooga & St. Louis Ry., Nashville, Tenn.

The line-brace casting serves as a packing ring, and also as an anchor for the chord to the cap. The chord members are packed only 1 in. apart to prevent the nesting of sparrows between them.

The four holes in the bottom part of the casting admit of fine adjustment and shimming where necessary, new wood being which to drive the anchoring spike being always available. This device has been found after long test to be entirely effective, even where heavy floods have overtopped the trestle and deposited driftwood against and under it.

The covering of the cap, instead of being continuous over the entire length as formerly, is made in three pieces, flaps being turned up on the sides of the chords under the overhanging sheet-metal chord cover. Fig. 3 shows the present standard open-deck trestle, and Fig. 4 the details of housing. These covers are fastened to the cap by nailing through strips of galvanized iron riveted to the underside of each cover piece at four points. This arrangement permits direct bearing of the stringers on the caps and ready removal of sheets for the purpose of adjustment. In place of two, the plans show three stringers on each side, which is now the standard on lines of heavy traffic. The addition of the third member to each chord was begun in 1905, and the iron applied in 1900 was found in many cases to be rusted out. The timber underneath the iron was found in as good condition as when covered in all cases where protection had been placed over new timber; but where timber had already been in place a few years before being covered the rotting begun in the open was found to continue under the cover. Upon removal of the covering all chord members beginning to show decay are marked with white paint on the underside and all defective covering replaced.

In the present plan the dowels are driven to a tight fit into the stringer instead of the tie, thus perforating the sheet iron upward instead of downward. Before driving the dowels the holes are filled with creosote, which is allowed to soak in. It was found necessary, in order to secure perfectly cut holes in the iron, to cut the upper end of the dowel in a lathe instead of by shearing.

After many experiments to determine the best way to prevent leaking through the dowel holes, it was found that this leaking was generally due to the churning of the ties and to capillarity, and that shearing the dowels was sufficient.

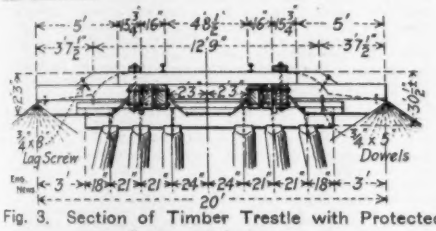


Fig. 3. Section of Timber Trestle with Protected Caps and Stringers.

(Showing also bulkhead for 20-ft. bank.)

The plan now in use is to bore, with an extension bit, a recess in the bottom of the tie concentric with the dowel hole ¼-in. deep and 2 ins. in diameter. Before putting the sheet iron on the stringers cut washers 1½ ins. in diameter are dropped over each dowel in the stringer. When the sheet iron is settled home by the weight of the ties, the washer forces it up into the recess in the tie, secures a better hold on the iron to prevent creeping, and to a great extent destroys capillarity by creating an open space instead of tight contact around the hole in the iron. The water must be forced up an incline in order to enter the hole.

The sheets of iron are connected to each other by bending and lapping and riveting at one point on each side of the joint where it overhangs the stringer. The sheets are ordered in such lengths as to bring the joints between instead of under the tie. This method has been found almost universally effective to prevent creeping, and has reduced the leaking very greatly. No remedy has been found for leaks around the guard-rail bolts, but all holes in the stringers are carefully plugged at the bottom after framing and filled with creosote which is allowed to soak in.

It is still found necessary occasionally to stop creeping by bending the iron downward on the inside of the chord, nailing it to the stringer and flaring the lower edge of the iron to keep the drip off the sides. Small squares of galvanized iron or washers are used when this plan is resorted to in order to keep the nail heads from pulling through the iron.

The method of covering the caps has proved entirely satisfactory; gum and red-oak caps in use five years show no signs of decay. No caps put on since 1900 and properly covered have required renewal. Caps next to the bulkhead and those subject to frequent overflow on low trestles are of creosoted pine, as housing does not prevent decay in these.

When the use of sheet iron on the stringers was begun in 1900, no attempt was made to secure metal of great durability. Failure of many sheets appeared in about three years at points where switching was done or much sand used. Under ordinary road traffic it lasted about five years, and on branch lines of light traffic even longer. No failures of the metal on the caps due to rust have yet occurred. Later on effort was made to secure greater durability, and after considerable investigation it was decided to use a charcoal iron made by an Eastern mill, the galvanizing upon which was found to be much heavier than on the ordinary galvanized (steel) iron purchased on the market. The difficulty with this iron was found to be that it

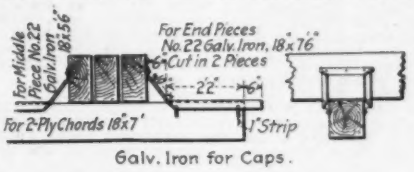

For Middle Piece No.22 Galv. Iron 18"x56"
For End Pieces No.22 Galv. Iron, 18"x76"
6" Cut in 2 Pieces
22"
16"
For 2-Ply Chords 18"x7"
1" Strip
Galv. Iron for Caps.
Bore in Ties 2 1/2" x 1/4" Holes for Washers and 7/8" x 3" Holes for Dowels. Drive Dowels to a Tight Fit in Stringers and place 3/4" Cut Washers around Dowels. Place Ties and Guard Rails. Jack up Ties and Guard Rail, place Galv. Iron over Dowels and lower them back to Position.
9'4"
No.22 Galv. Iron
For 3-Ply Strs. 34 1/2" 9'0", 2" 24" x 9'0"
On Ballast Floors, Omit Dowels
26"
3/4" Cut Washer
Eng. News
Galv. Iron for Stringers

Fig. 4. Details of Application of Galvanized Iron to Stringers.

would not stand short bending, the cracking causing leaks.

Ballasted floors on trestles are meeting with much favorable consideration at present on account of the low cost of maintenance and decreased danger from fires, but it has not been heretofore considered advisable to use this plan without treating piles, caps and chords.

On account of the success attained in the preservation of chords by housing, a plan for ballasted floor similar to that shown in Fig. 5 has been adopted on the lines of which the writer has charge. The creosoted flooring on top of the stringers is confined to place by lag screws passing upward through a casting which also engages the packing bolts passing through the upper part of the chord. By this method all perforation of the chord covering is avoided. The chord covering can be renewed when required by jacking up the flooring from the caps below, and at the same time the members of the chord can be inspected on top. The cap covering is the same as for the open deck. This plan is much cheaper than one consisting of a larger number of creosoted stringers with plank spiked on top of them, and has the advantage of immediate conversion of present open-deck trestles to ballasted floors and admits of more ready inspection. It is believed to be able to take care of derailed equipment as successfully as any other plan. Driving spikes into creosoted timber penetrates the outer surface and admits moisture to the untreated interior, and will no doubt hasten the decay of the timber.

It requires 7⅛ sq. ft. of the galvanized iron to cover caps and stringers for every lineal foot of trestle with 12-ft. spans. The best quality of iron will cost about 0.042 cts. per sq. ft. or 0.308 cts. per lin. ft. of bridge. The labor cost for open decks, including the boring of recesses in the ties, is about 0.087 cts. per lin. ft. of bridge, making the total cost of covering per lin. ft of bridge about 0.395 cts.

If the stringers and caps of open-deck trestles were creosoted instead of covered, the extra cost for treatment

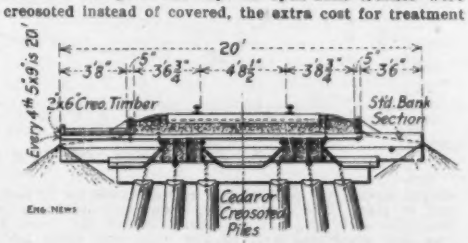


Fig. 5. Section of Ballasted Floor on Timber Trestle.

would be about $10 per thousand, B. M., or 75 cts. per ft. of bridge. To secure the full benefit of treatment, the framing should be done in advance, which is often quite difficult, in view of the varying span lengths, and cutting has often to be resorted to. If not done in advance, decay is likely to set in. Assuming that creosoted stringers will last 20 years, and that the galvanized iron on untreated stringers will last only five years, it would seem the best policy to use the creosoted timber. On the other hand, the creosoted timber should be protected against fire, and a covering which will answer this purpose will also protect the timber from decay.

For ballasted deck trestles the protection from fire is afforded by the ballast, and for all new work there seems no doubt but that the best plan is to use creosoted stringers and caps. However, for present conditions, where all stringers are already in place and sound, it seems most economical to cover the present stringers and to continue to renew the covering until new stringers are required. All patching should be done with creosoted stringers. Just what effect the placing of the creosoted timber on top of the galvanizing covering is likely to have on the durability of the latter is a matter for future determination.

It is also a question whether black iron painted with two coats of red lead applied immediately after mixing would not be more economical than the galvanized iron for the open-deck trestle.

Resistance to the Flow of Water Through Locomotive Water Columns.

Prof. A. N. Talbot and Mr. M. L. Enger, of the University of Illinois, have recently completed a series of tests on the hydraulic properties of what are known as water columns, for delivering feed water to the tanks of locomotives. The results of these tests are recorded in Bulletin 48 of the University of Illinois Engineering Experiment Station, from which the following information is taken. The tests were on 14 water columns, including the principal types used in the United States. It was intended (1) to determine the loss of head through the columns, including the loss in the valve, and in the riser and spout, and (2) to note the action of the valve during the closing and the resultant pressure changes in front of the valve to find the influence of the valve on the development of water hammer in the supply line. These tests were made in cooperation with the Committee on Water Service of the American Engineering & Maintenance of Way Association and others.

The resistance to flow was found to be surprisingly high, much higher than has previously been believed. With a discharge of 4,000 gals. per min., the loss of head ranged from 15.4 to 46.5 ft. The water columns giving the higher figures included types which are used in large numbers by various railroads. The loss of head in the case of the water column showing the highest resistance is equal to the friction loss of a 10-in. straight pipe line 566 ft. long. It is also 25 times that through a single 10-in. elbow. As a result of the tests, it was stated that the discharge ratings of most water columns is too high, that is, the discharge under working conditions is much smaller than has been generally claimed.

Examination of the mechanical construction showed that the form of valves and passages, in the cases of high resistance, did not accord with the principles of good hydraulic design. Such objections were found as tortuous passages, sudden change in direction, and sudden contraction and expansion of section. Good hydraulic design was found to have been sacrificed for convenience of construction, inspection and repair. However, modifications of mechanical design, suggested since the tests, in some cases have improved the hydraulic conditions without impairing the mechanical design.

The telescopic adjustable spout showed lower losses than the fixed spout, though the lift was greater with fixed spouts as the point of discharge with adjustable telescopic spouts was taken as the end of the riser. The ball-and-socket spout shows about the same friction loss as the rigid type, although the point of discharge was usually lower. "Anti-splash" attachments were found to use up head in providing a solid stream.

In all but two of the types tested, only 15% of the valve movement was effective in giving high resistance to prevent hammer, and in all but one the resistance through the valve opening was not appreciably larger than at full opening until the valve was 85% closed. It was recommended that time gained in quickly making 85% of closure be applied to lengthening the time for the remaining 15% to bring the moving column to rest slowly and prevent water hammer.

Collapse of a Steel Water Conduit From External Air Pressure While Under Test; Bull Run Pipe Line, Portland, Oregon.

By J. C. STEVENS,* Assoc. M. Am. Soc. C. E.

Bull Run Conduit No. 2 is now being laid to provide an additional water supply for Portland, Oregon. This consists of 24 miles of 52 and 44-in. lock bar pipe, laid parallel, for the most part, with the old Bull Run Pipe Line† (Conduit No. 1). In the course of field tests of pipe, in the trench, two sections have developed weakness, and, as a result of the sudden escape of the water, sections of pipe near the summits of the sections under test have collapsed from external air pressure. A third collapse occurred while water was being drawn off after a successful test had been made.

From the headworks on Bull Run River to Lusted's Hill, a distance of 52,915 ft., the pipe is 52 ins. in diameter and varies from ¼ to 7/16-in. in thickness. The hydraulic gradient is 1.691 ft. per 1,000 from elevation 746.0 at the headworks. From Lusted's Hill to Reservoir No. 5, on the south slope of Mt. Tabor, a distance of 75,379 ft., the pipe is 44 ins. in diameter and ¼ in. in thickness. The hydraulic gradient is 3.240 ft. per 1,000 from elevation 656.5 to elevation 411.6 at Reservoir No. 5. The capacity will be approximately 45,000,000 gals. per day, or practically twice the capacity of the old line, and will virtually exhaust the natural low-water flow of Bull Run River at this point.

The contract for furnishing materials and laying the pipe was awarded to the Schaw-Batcher Co., of Sacramento, Cal. The lock-bar pipe is being furnished to the contractors by the East Jersey Pipe Co., of New York City.

SPECIFICATIONS.—The specifications pertinent to this paper are quoted below. They were primarily prepared with a view of using riveted steel pipe, but paragraph 33 admitted the use of lock-bar pipe.

(3) Quality of Steel.
The steel plates used in the manufacture of the pipe shall be made by the open-hearth process, and shall be of the grade known as "soft," and must comply with the following requirements:
The ultimate tensile strength of the steel shall be between 52,000 and 62,000 lbs. per sq. in. Test pieces to be 8 ins. long and 1½ ins. wide between measuring points. The elastic limit shall not be less than 31,000 lbs. per sq. in. The elongation shall not be less than 25% in 8 ins. when measured longitudinally, and 22% when measured transversely, and there shall be a reduction of area at the point of fracture of not less than 50%.
(4) Bending Test.
The bending-test specimens shall be cut lengthwise and crosswise from the sheets, and shall not be less than 6 ins. long and 1 in. wide. When cold they shall be bent 180° upon themselves and hammered down flat, without showing any signs of fracture on the convex side of the bent portion.
(5) Punching Tests.
The specimens for punching tests shall be 1⅝ ins. wide and not less than 10 ins. long; a row of not less than eight holes, ¾-in. in diameter, spaced 1¼ ins. between centers, shall be punched while the plate is cold without causing any cracks.
(6) Drifting Tests.
Specimens for drifting tests shall be 3 ins. wide and not less than 5 ins. long; not less than two holes ¾-in. in diameter, spaced 2 ins. between centers, and 1½ ins. from the edges, shall be punched and then enlarged cold by blows from a sledge hammer upon a drifting pin, until said holes are at least 1¼ ins. in diameter, without causing any cracks.
(7) Chemical Tests.
Chemical analysis of each heat or melt will be made by experienced chemists, under the direction of the engineer or his inspector. The steel must not contain more than 0.05% sulphur, 0.04% phosphorus, 0.10% silica, and 0.5% manganese.
(8) Additional Tests.
The plates must also admit of cold hammering, and of scarfing to a fine edge at the laps, without cracking. The test pieces must furthermore stand quenching, forging and other tests as may suffice to exhibit fully the temper, soundness and fitness for use of the material. The failure of said test specimens when taken at random, from the finished products of any heat or melt, to conform with any or all of the above stipulated requirements, will be sufficient cause for the rejection of the entire product of such heat or melt.
(9) Perfect Plates.
The plates shall be free from laminations, cinders, and any other surface defects. They shall be fully up to the required thickness at the edges, and any plates which shall be found to be more than 5% short of the required thickness at any point, shall be rejected. Not over 5% of the plates shall be short of the full required thickness at any point. The plates must be rolled flat and sheared as accurately as possible (but in no case shall any plates be short of the specified dimensions), and must in all respects be in good merchantable condition.
(11) Rejections.
Failure of specimens to fully and satisfactorily conform with the requirements of these specifications when subjected to the tensile, punching, bending, drifting, chemical and other tests required, or to any of said tests, shall be followed by the rejection of the entire heat or melt from which the said samples were obtained. Should any sheet or plate show defect during the process of punching, bending and riveting, for manufacture into pipes, it shall be rejected, notwithstanding the test pieces from the heat or melt from which said plates were manufactured may have previously been satisfactorily tested.
For the purpose of excluding inferior metal, 20% of the top of the ingot shall be cut off and rejected, and if the remaining metal shows evidence of not being perfectly sound, further cuts shall be made.
(18) Manufactured Pipe.
The plates must be of such dimensions as to admit of being rolled into shapes that will produce what is known as the taper-joint pipe, the small end of each section having the full inside diameter called for in each size of pipe. The longitudinal seams will alternate to the right and left, but not more than 6 ins. from a true line along the axis of the pipe, and when laid, they must be on the upper half of the pipe.
(21) Shop Testing.
Each section of the pipe after completion and before being coated, shall be subjected to a hydrostatic pressure of at least one and one-half times its normal working pressure as shown on attached profile of pipe line, and must, during said test, be absolutely free from leaks or fracture. Unless otherwise directed by the engineer, all pipe for use under 50 lbs. pressure or less shall be tested to 75 lbs. per sq. in.; all pipe for 50 to 100 lbs. pressure shall be tested to 150 lbs. per sq. in.; all pipe for 100 lbs. pressure and above shall be tested to 225 lbs. per sq. in.
(25) Air Valves.
Air valves 4 ins. in diameter are to be placed on the pipe at points indicated on the profile. The valves are to be of the pattern known as the "Crispin Automatic Air and Vacuum Valve," or some other approved model, with ¼-in. bibb. Cast-steel branches, having flanges to conform to the circumference of the pipe at one end, and the other end to fit the flanges of the valve, shall be used for attaching them to the pipe. Between the check valve and the casting on the pipe there shall be placed a 4-in. double-flanged gate valve. Each air valve is to be protected by a concrete well and iron cover, as shown on the plan. Where the pipe is located within 30 ft. of the hydraulic grade line, galvanized-iron stand pipes with 3-in. service pipe near the pipe may be substituted for the air valve, if directed by the engineer. These stand pipes to be boxed in with a box filled with sawdust.
(26) Blow-Offs.
Blow-offs 8 ins. in diameter are to be placed at the bottom of the pipe in depressions as shown on the profiles. Each blow-off shall be furnished with sufficient 8-in. extra-heavy, kalamein pipe, properly laid to point of discharge.
(32) Field Tests.
After the pipe has been laid in the trench it must be plugged at the ends and tested with hydrostatic pressure in convenient sections, of such length as the engineer may direct, to at least 1½ times the pressure to which it shall be subjected under the natural working pressure when in operation. Calking must be continued until the pipe is tight at this pressure. Unless otherwise directed by the engineer, all pipe used under 50 lbs. pressure or less shall be tested to 75 lbs. per sq. in. All pipe used under 50 lbs. to 100 lbs. pressure shall be tested to 150 lbs. per sq. in., and all pipe used under pressure exceeding 100 lbs. pressure shall be tested to 225 lbs. per sq. in.
The contractor must find his own ways and means and all labor and appliances and material for testing, and the caps, plugs, etc., necessary for closing all openings in the pipe and branches. The water required for this purpose will be furnished by the Water Board from the present conduit, at such times as other demands upon the conduit will permit, free of charge, but the cost of tapping the main and the necessary pipe must be borne by the contractor.
The closing of pipe ends and all openings must in all cases be absolutely tight, to prevent filling the trench with water. Should any pipe, special or appurtenances break when making said tests, they must be replaced by the contractor at his own expense. On the completion of the main and its connection with the city reservoir, the contractor shall maintain the pipe in good order for the first six months of continuous service, and shall, at his own cost and expense, repair all damages or leaks which may develop during that period. All expense of said tests to be borne by the contractor.
(33) Lock-Bar or Welded Pipe.
Lock-bar pipe or pipe with welded longitudinal seams may be used, subject to the approval of the engineer. Detailed plans and specifications shall be furnished by the bidder showing wherein the pipe it is proposed to lay differs in workmanship from the pipe described in the foregoing specification. The size, length and thickness of pipe for the different sections and the number and dimension of manholes, air valves, blow-offs and all classes of special construction and protection shall conform to these specifications.

The specifications for coating with "Mineral Rubber Coating" were published in Engineering News, April 28, 1910.

Fig. 1 shows some of the lock-bar pipe at the storage yard on Lusted's Hill. They are made in 30-ft. lengths. Fig. 2, taken at station 1108, illustrates the method of laying and joining. The pipe is laid so that the bars are approximately on the horizontal diameter. Fig. 2 also shows the concrete and steel trestles used and a blow-off in place. Fig. 3 is a condensed profile of the line.

METHOD OF MAKING FIELD TESTS.—The selection of sections to be tested is optional with

FIG. 1. LOCK-BAR PIPE FOR BULL RUN LINE NO. 2, IN STORAGE YARD.

FIG. 2. METHOD OF LAYING AND JOINING LOCK-BAR PIPE ON THE BULL RUN LINE.

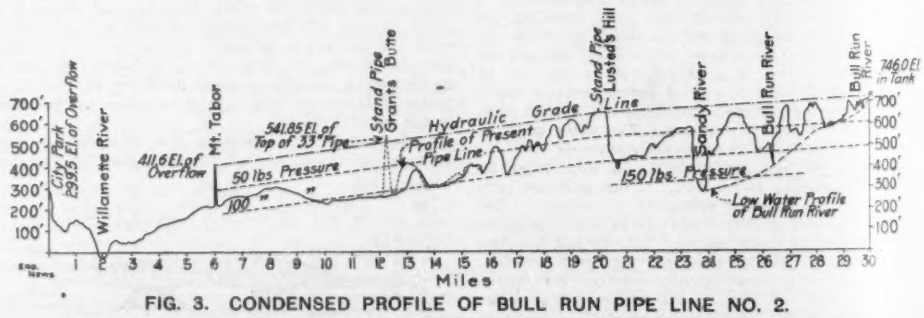


FIG. 3. CONDENSED PROFILE OF BULL RUN PIPE LINE NO. 2.

*Civil and Hydraulic Engineer, 605 Spalding Bldg., Portland, Oregon.

†See "The Water Works of Portland, Ore.," by W. P. Hardesty, C. E. Eng. News, Feb. 6, 1908.

the contractor. A section is chosen, varying from 4,000 to 10,000 ft. in length, and a "dead end" is bolted to each end of this section. The "dead end" is dished inward and is also supported by blocking against the adjoining section, or by bracing. The pipe is then filled with water until it overflows at the air-valve branches on the summits. The gate valves in these branches are then closed and the air valves are bolted in place. In some instances the air valves were placed before water was admitted, and in some cases no valves were used. When all is ready, additional water is pumped into the pipe by means of a portable, steam pump and boiler, until the desired pressure is reached. This pressure is then held by slow pumping until the section under test is thoroughly inspected and all joints that show leakage are calked.

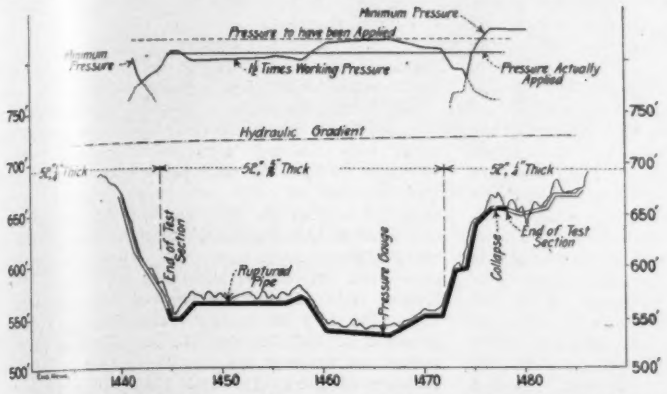


FIG. 4. PROFILE AND HYDRAULIC PRESSURE LINES OF SECTION 1440-1480, BULL RUN PIPE LINE.

The actual pressure to be applied in each section is determined by the Engineer of the Water Board, since the specifications (paragraph 32) fix the limits only within which the pressure must be applied. It is, of course, physically impossible to apply exactly 1½ times the working pressure and at the same time observe the minimum of 75 lbs. per sq. in. The usual procedure is to apply 1½ times the working pressure to the lowest point in the section, provided this gives a pressure near the minimum at the summit. If not, the minimum on the summits is fixed as the governing factor, and if this gives too great an excess on the lowest portions, the contractor is of course at liberty to subdivide the section. The maximum pressure of 225 lbs. was fixed to take care of the Sandy River crossing. The maximum head at this point is 456 ft., corresponding to a working pressure of 198 lbs. per sq. in., and would have required a pressure of 297 lbs. if the maximum of 225 lbs. had not been adopted.

On the earlier tests only a small amount of back fill was placed in the trench before the test. Now the practice is to back fill completely to grade in the center of each section of pipe, leaving the joints exposed for inspection.

As already noted, in the course of the field tests two pieces of the lock-bar pipe broke during the test. Each break brought about a collapse of the pipe near the summits of the test sections, due to unbalanced atmospheric pressure outside following the rapid emptying of the pipe. A third collapse occurred while draining a section after a successful test.

BREAK AT STATION 1452.—On Nov. 20, 1910, a section between Station 1445 and 1478 was under test when one of the lock bars in a pipe at Station 1452 split, resulting in the collapse of two sections of pipe at Station 1476. Fig. 4 shows the profile for the section under test, the location of the pipe that burst, and those that collapsed.

Above the profile (Fig. 4) are also shown the hydraulic gradient, the line of pressure necessary to conform rigidly to the specifications, the line of pressure adopted for the test, and the line of pressure actually applied when the pipe burst. Pressure lines are represented in feet of hydraulic head with the pipe line as base. The head corresponding to the minimum pressure of 75 lbs. per sq. in., is 173 ft., and this pressure line governs the summits until it crosses the line of 1½ times working pressure at Stations 1442 + 30 and 1474 + 40. The pressure gage was placed at Station 1466. The pressure to have been applied was 125 lbs. at the gage, but the pipe at Station 1452 burst when the gage registered 120 lbs. The pressure on the pipe at Station 1452 was 107 lbs., at the time of failure. The pipe is 52 ins. in diameter, made of 5/16-in. metal. Hence the static pressure causing rupture was 2,780 lbs. per linear inch along the bar. The stress on the metal plate was 8,900 lbs. per sq. in.

The rupture was due to the lock-bar splitting, letting the plate pull out of the bar. The plate itself was not injured, neither did the upset on the edge of the plate show signs of shearing. The plate pulled out of the bar for practically half its length, or 14 ft., and pulled out the rivets for a distance of 10 ins., on the circumference of the joint between this and the adjoining pipe. After failure, a longitudinal crack was found on the inside of the jaw of the bar and on the convex side of the plate, as shown in the section in Fig. 5. There were no transverse cracks in this bar.

Two sections of pipe and about 3 ft. of a third section collapsed in a horizontal plane between Stations 1476 and 1477. The rivets in the joint of the partially collapsed pipe were pulled out, admitting air to the pipe, which prevented further collapse. The trench at the partially collapsed pipe and all other portions near this summit (except at the two sections that collapsed totally) were fairly well back filled.

COLLAPSE AT STATION 1014.—A collapse of 7½ sections of 44-in. pipe, ¼-in. thick, occurred between stations 1013 + 20 and 1015 + 60 on Jan. 9, 1911, while the section between stations 970 and 1073 was being emptied after completion of a test. The water was being drawn off from a tap in the "dead end" at station 970, in a 2-in. pipe, leading into the adjoining section.

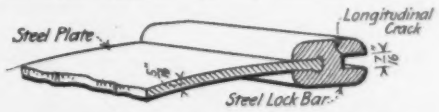
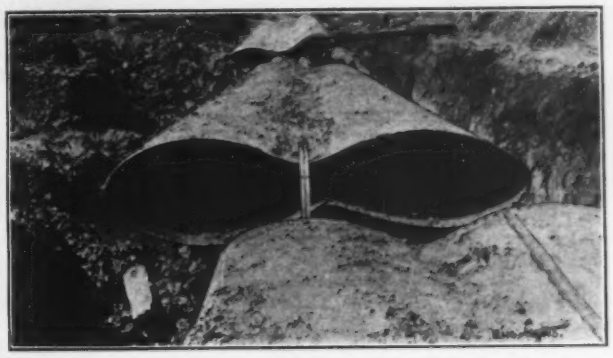


Fig. 5. Section of Cracked Lock Bar.

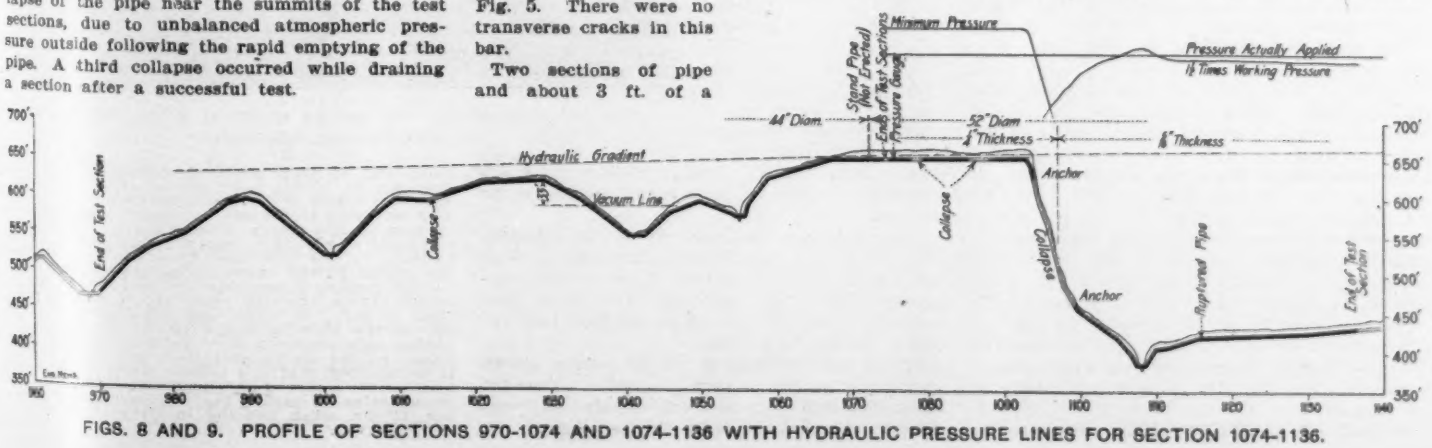
The water had been running for two days when the collapse occurred. No air valves had been placed, and the gate valves in the air-valve branches were carelessly left closed after the test was finished. The workman who was drawing off the water was told to open the gate valve but he forgot to do so. He has not been seen since the collapse occurred.

These pipes collapsed in a vertical plane. Fig. 6 shows seven of the collapsed pipe after they had been removed from the trench. The eighth section, being partially collapsed, was straightened and replaced in the trench. Fig. 7 is a closer view of three of the pipe shown in Fig. 6, and shows the position of the bars with reference to the plane of collapse. Fig. 8 shows the profile of the section under test and the position of the collapsed pipe. In every pipe there is a 1-in. tap near one end, closed by a 1-in. screw plug. At the upper end of the section under consideration a few of these taps were open while the water was being drawn off. These would provide air for the pipe from the summit at Station 1029 to the upper end of the test section at Station 1073.

This collapse differs from those caused by the bursting of a pipe, in that the external pressure increased gradually. Nevertheless it is practi-



FIGS. 6 AND 7. COLLAPSED PIPE AT STATION 1014, BULL RUN PIPE LINE, PORTLAND.



FIGS. 8 AND 9. PROFILE OF SECTIONS 970-1074 AND 1074-1136 WITH HYDRAULIC PRESSURE LINES FOR SECTION 1074-1136.

FIG. 10. COLLAPSED SECTION OF BULL RUN PIPE LINE AT LUSTED'S HILL.

cally certain that those portions between the summits at Stations 1019 and 1029, where the trench had been partially back filled, sustained an external pressure of practically one atmosphere without collapsing. Over the portion that did collapse there was less back filling.

The statement has been made that this collapse occurred under a very low external pressure—2 or 3 lbs., per sq. in. That this was not the case is evident from a study of the profile (Fig. 8). As the water was gradually drawn off, it receded from the upper end to a point near Station 1059, the elevation of which is the same as the summit at Station 1029, before there was any external pressure. As emptying continued, an external pressure would gradually develop throughout the remaining portion of the pipe that lies above the line marked "vacuum line" on the profile. The column of water would not be broken until the water receded to this line at Station 1048, whose elevation is 33 ft. (hydrostatic head corresponding to a complete vacuum) below the summit at 1029. When this occurred, the column would break at Station 1029, forming a complete vacuum.

The summits at Stations 990 and 1010 are at the same elevation. A complete vacuum would not form at these points until the receding water reached them. When this occurred, the column of water would break at Station 990, leaving equal columns on each side of the depression at Station 1001. While the water was slowly receding from Station 1029 to 1010, sufficient air doubtless leaked into this portion of the pipe to produce an oscillation at the time the column of water broke at Station 990. This oscillation was doubtless the immediate cause of the collapse at Station 1014. The difference in the amount of back fill at the collapsed portion and those adjoining was very slight. In no case was the pipe entirely covered over, the earth merely extending upon the sides to about the middle of the pipe.

BREAK AT LUSTED'S HILL.—On Feb. 19, 1911, a section between 1073 + 84 and 1136 was being tested when a 5/16-in. pipe burst at Station 1116, causing the collapse of 22 sections of 52-in. pipe near the summit of Lusted's Hill. Fig. 9 shows the profile of the section under test and the position of the collapsed pipe. Above the profile have been drawn pressure lines corresponding to rigid requirements of the specifications. The pressure actually applied was less than 1½ times the working pressure at the lowest point, or 60 lbs. at the summit, instead of the minimum of 75 lbs. The pressure gage was located at Station 1074. The desired pressure, 60 lbs., had been reached and the process of inspecting and calking had begun, when the pipe at Station 1116 burst. The city inspector was in the trench near Station 1093. He felt the shock when the bar broke, climbed out of the trench onto the pipe and turned around in time to see the collapse occur.

Fig. 10 is a view of the collapsed sections from the summit near Station 1094, looking down the hill. This was taken the day after the collapse occurred. As seen from the profile (Fig. 9), this hill is much steeper than would at first appear from Fig. 10. This collapse, as shown in Fig. 10, was in a vertical plane and extended between two anchors, placed at Stations 1093 + 79 and 1096 + 29, respectively. These anchors consist of a heavy steel band bolted to a concrete base shaped to fit the lower portion of the pipe. The upper anchor band shows plainly in the figure, and the lower one is at the end of the collapsed portion. It will be noticed that there was practically no backfilling in the collapsed portion, while farther down the hill the backfilling partially covered the pipe.

Fig. 11 is an end view of two of the collapsed pipes shown in Fig. 10, after removal from the trench. The relative position of the bars and the plane of collapse is plainly shown.

The two other collapsed portions were on the summit from Stations 1077 + 84 to 1079 + 04, and from Stations 1085 + 34 to 1088 + 04. Both of these sections collapsed in a horizontal plane. The amount of back filling at these sections was less than in the remainder of the trench on the summit.

Figures 12 and 13 show two of the collapsed pipes from near Station 1087 after removal from the trench. The one in Fig. 12 was turned completely over when removed. This view also shows the rigidity with which the bars held the plates during the collapse.

At the time of collapse the air valves, shown on the profile, Fig. 9, were in position. The pipe at Station 1095 + 92 in which the air valve was located, collapsed as badly as the others.

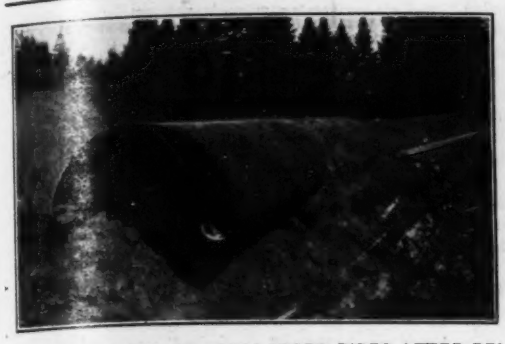
The bar in the pipe at Station 1116 was under a head of 370 ft., corresponding to a pressure of 160 lbs. per sq. in., when the rupture occurred. The tension was therefore 4,160 lbs. per lineal inch along the bar, and the pipe was under a tensile stress of 13,300 lbs. per sq. in. The bar failed both transversely and longitudinally. The plate pulled out of the lower jaw in the bar along its length of 14.2 ft. The upset edge of the plate showed no evidence of shearing. Fig. 14 is a view of the ruptured pipe after its removal from the trench. As will be seen, nine rivets were pulled out along the circumference of the joint between this and the adjoining pipe. The inside of the jaw on the convex side of the plate was cracked longitudinally the entire length of 14.2 ft., in the manner shown in Fig. 5. The position of the transverse cracks are sketched in Fig. 15. The bar was broken entirely across at A and B., while the others showed on the inside of the pipe only. Two test sections have been cut out, as shown in the sketch. The broken ends of the bar showed crystallization. This pipe had previously been subjected to a pressure of 157 lbs. per sq. in. Earlier in the season an attempt was made to test a section between stations 1110 and 1140. The pressure gage was placed at 1139, and a pressure of 155 pounds was to have been applied at that point. When the gage registered 150 lbs. the "dead end" at Station 1140 "dished" outward and the test was postponed until the pipe had been laid to the top of Lusted's Hill. In view of this experience, the contractor objected to putting a "dead end" in the pipe on the steep slope of Lusted's Hill for fear the trench would wash out in case of a break there.

After the break and collapse of Feb. 19, however the pipe was replaced and successfully tested in two sections with the "dead ends" at Station 1097, where the thickness of pipe changes from ¼-in. to 5/16-in.

AIR VALVES.—The specifications call for "Crispin" air valve, or some other approved model. Several makes of valves have been experimented with, but those in use during the collapses (where any at all were used) were the "Simplex." These do not have a clear opening of 4 ins. Several have been measured and found to have an opening at the inlet of about 7 sq. ins., instead of 12.5 sq. ins., that would obtain with a 4-in. circular opening. It is doubtful if any air valve of this capacity could have prevented the collapse at Lusted's Hill.

FIG. 11. COLLAPSED PIPE AFTER REMOVAL FROM TRENCH, LUSTED'S HILL SECTION.

A Turbine Locomotive has been successfully tried at Milan. The peculiar feature of the turbine is the use of movable blades, which are operated in series. Four sets of such blades are used, and at high speed the steam strikes the first set of blades only, while at intermediate speeds two sets or three sets can come into play. The reversing mechanism is a special and unique feature of this motor. The rotors have two sets of blades which are of opposite curvature. When running in one direction the steam passes over the blades at the outer circumference from left to right; when running in the opposite direction steam passes over the other set of blades from right to left. In either case the loss of energy due to the blowing action of the second set of blades only amounts to a small fraction of the total, and the experiments show it to be 2 to 2½. It is reported that this engine starts well under load both on curves and gradients, and that the consumption of steam has not exceeded 38 lbs. per HP.-hr. when running in either direction.—"Engineering."



FIGS. 12 AND 13. COLLAPSED PIPES AFTER REMOVAL FROM TRENCH NEAR STATION 1087, BULL RUN LINE.

A Strong Recommendation for Direct City Construction Instead of Contract Work at New Orleans.

A well-put argument for direct city construction of an extension of the drainage system of New Orleans, instead of letting a contract for the work, was made recently by Mr. George G. Earl, M. Am. Soc. C. E., General Superintendent of the Sewerage and Water Board of New Orleans. Legal obstacles, whether real or apparent we cannot say, were found by someone to prevent the carrying out of Mr. Earl's recommendation. The report (dated May. 25, 1911) is none the less interesting on that account, and is given here in full:

In connection with the construction of the Napoleon Ave. [drainage] canal, about to be undertaken, the question has arisen in the engineering department whether this work should be done by general contract or whether it should be done by contracting for supplies, material and machinery, and the construction, installation and erection of same by forces employed by the Board.

Experience to date indicates that the Board's forces can conduct and organize any construction work as well as any contractor can organize it; that the Board can buy as good or better material by contract as cheap or cheaper than a contractor can buy it, and that it can be more certain of getting the character of material and supplies required for the best interest of the work if it is the purchaser direct instead of the general contractor who profits every time he can use a cheaper material.

FIG. 14. RUPTURED PIPE TAKEN FROM TRENCH NEAR STATION 1116, BULL RUN LINE.

In the drainage construction, the following consideration will show the possible saving to the Board by the use of the small contract and force account system rather than the general contract system:

The contractor has to make a bond in a surety company in an amount equal to 25% of his contract. He has to put up a lot of security or collateral to obtain this bond and to pay a high premium for it. Unless he is very strong financially this forces him to borrow money to prosecute his work. He has to pay interest on borrowed money and on deferred payments on materials, etc., and to await the completion of his work to get his retainer or final payment, which is 15 to 20% of the amount of his contract; so that, all told, it will cost him from 5 to 10% of the amount of his contract to bond and finance it. He will bid for a higher profit, but assume he only makes 10% and you have a cost to the city above the actual cost of the work of from 15 to 20%.

The contractor will also need a lot of special plant for the execution of his work. He will figure that this plant may never be of any use to him after this particular contract is completed, in which case he must figure the cost of his plant into the cost of the work.

The contractor will understand that the drainage canals to be constructed are dependent upon the existing drainage stations to be kept clear of water; that these stations themselves will be under enlargement at the same time; and if the contractor assumes no obligation, for his convenience he will add a lot to his price for the risks of working in canals which are already in operation and through which the drainage of the city must pass. If the contractor does assume any obligations on the part of the Board to do anything specific in relieving his work of water, he will claim that the Board has not lived up to that obligation, and seek payment for extra work and damage due to its failure. With two or three contractors working, as they must be, so that any action at the pumping station which helps one will injure another, there will be still more confusion, and each contractor will be morally certain that he is getting the worst of it.

In the designation of spoil banks for waste excavation from the canals, and the disposal of same, there has always been contention as to the location of such spoil banks, and as to the shifting of them, which frequently becomes necessary, due to one cause or another; and in the Napoleon Ave. canal system the question of exactly how the dirt will have to be handled is one of the serious problems to be worked out as the construction advances. Where the Board controls the work directly it can gain advantage by selling filling and shifting arrangements to meet conditions as they arise; where it is attempted to handle this question through a contractor, claims for extra work and damage for delay, etc., arise every time anything like a change or delay for this cause arises, and many of the claims and contentions that have arisen on former drainage contracts have been due to such causes.

On large contract work the forces of the contractor are handled to get quantity of work, and the Board's forces are there to insist upon quality, and the total of both forces has to be paid for by the Sewerage and Water Board, and this total is larger than it need to be if there is no antagonistic interest of this character to be taken care of.

These works are large and important and the men in charge of them can do far better work in proportion, as all kinds of friction due to opposing interest are eliminated; so that they can devote their whole ability and energy to getting the best results at the least cost.

The Board possesses the following advantages in getting cheap and good work. It has no bond, interest on borrowed money or retainer to consider. The contractors' profits are eliminated. It can buy the best material and supplies for the intended use without the restraining influence of the general contractor whose interest lies in getting the cheapest. It only costs the Board 1% in loss of interest on its funds as they are expended. Machinery and plant for construction of one work can be transferred to another of a similar kind indefinitely as works are completed, thus only costing its actual depreciation for any particular work. The Board has rights and advantages and consideration from other public departments of the city and from public-service corporations which it cannot delegate as fully to its contractors. The Board has a wharf at the water-works intake, and land, connected with the City Belt Railroad over which supplies by river can be received. It has its great Washington Avenue Yard connected with the Illinois Central Railroad and centrally located to the greater portion of the proposed drainage work, at and through which certain of its supplies and services can be obtained; and in many other ways has organization and advantages necessary for other of its functions which can be utilized for any of its work but the utilization of which would not be possible to be made available to its contractors, and which afford facilities that each contractor must obtain for his own particular work, and which will cost him more than the pro-rated cost of equal or better facilities to the Board.

The work of sewerage construction undertaken in 1908 by the Board because of bids then reported to be 30% too high has given a good example of what the forces of the Board can do in this respect. To Dec. 31, 1910, 80.5 miles of sewers were constructed in this way, and the cost to the Board for the work was not over $676,000, including all labor, material, supervision, depreciation on tools, machinery, wagons, stock, etc. Exactly the same items constructed at the prices bid by contractors would have cost $1,055,415. No one knows how much extra work would have been claimed, but 3% for extra work and 5% for supervision in the field are about the average payment or cost of seven contracts immediately preceding these, or 3% in all, making $84,433 to be added, making $1,139,849 which this work would have cost under these contracts, without including the pro-rating of general expense of $30,000 for two years, which is included in the $676,000 above stated as cost of the work done by the Board, which general expense would have been about the same in either case and must be added to the one or deducted from the other for comparison, making $676,000 against $1,169,849, or a difference of $494,500, where a saving of only 30%, or about $338,000, was forecast, the actual saving being 42.5%.

If the average of the prices in Contracts "U," "V" and "W" (the last sewer contracts constructed by Irwin Bros.), which prices were considered conservative, be applied to the quantities of work actually done by the sewer construction forces of the Board, we have the following result:

Cost of work under items	$797,309
Extra work (3%)	23,919
Supervision in field (5%)	39,866
General expenses (two years)	30,000
Total	$891,094

The bids rejected in September, 1907, were nearly 25% higher than these bid prices. The actual cost of the work done by the Board's forces shows a saving of $215,760 over even these figures, a saving of more than 24%, and these figures are considered to be reasonable bids.

That any such saving as above indicated could be made in the Napoleon Ave. system over conservative contract bids, there is no claim. Your General Superintendent believes, however, that a saving of certaintly 10%, and possibly of 15% or even more, can be so effected; that better work at less public inconvenience can be obtained,

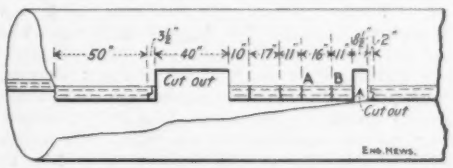


Fig. 15. Sketch of Ruptured Pipe and Lock Bar No. 97,154, at Station 1116.

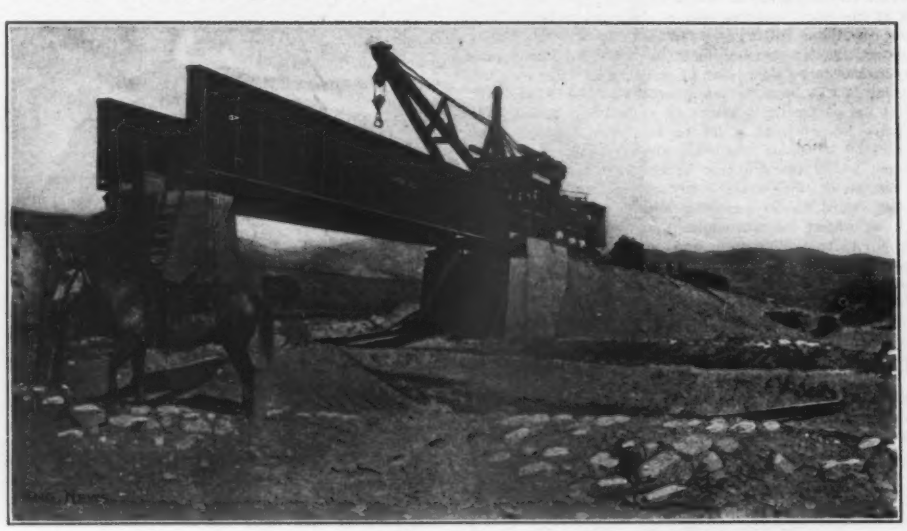
and must therefore so report. No member of the Sewerage and Water Board has ever breathed a suggestion that this work be done in this way, but the idea as to this canal has originated from a recommendation to me by Mr. Kirkland, who has had all of the experience of all of the drainage work to this time to base his judgment upon.

Your General Superintendent recognizes that if improper men, or useless men, were employed on the construction the results would be reversed, and would not join in this recommendation if his experience to this time led him to fear any such danger, but having had eleven years of experience with the conservatism and firmness of the Sewerage and Water Board and its officers in conducting its affairs, with a single view of the best interests of the city, he has no hesitation in recommending that this work on the Napoleon Ave. lined and covered canal system be started and tried out along the lines indicated, provided this course be not in conflict with the law.

Work on the Jungfrau Railway Extension has progressed so that the stretch from Eismeer station to Jungfraujoch is nearly completed. About 80% of this 2.4-mi. section is already equipped with track and electric conductors. The elevations of the two endpoints of the section are 10,350 ft. and 11,220 ft., with a gradient of 6.6%. Toward the upper end a 500-ft. lateral drift has been opened to the cliff face, to dump spoil; this drift was holed through on June 15, 1911. Beyond station Jungfraujoch will follow a 25% grade to the summit station at El. 13,680 ft. above sea level. —(Ztschr des Ver. Deutscher Ingenieure, July 8, 1911).

A Special Derrick-Car for Setting Girder Spans; San Pedro, Los Angeles & Salt Lake R. R.

In reconstructing the San Pedro, Los Angeles & Salt Lake R. R. through the Meadow Valley Wash, between Moapa and Caliente, Nev., which was wrecked by flood on Jan. 1, 1910, there are required 15 crossings of the channel, involving 31 steel spans 90 to 150 ft. long. Temporary by-pass lines are used by the railway pending the erection of the bridges; running falsework is generally avoided. The erection of the bridges is being done by the engineering and contracting firm, Shattuck-Edinger Co., who also contracted for grading the westerly 36 mi. of roadbed and for building the substructures of eight of the bridges. This firm has constructed a rather special derrick-car, shown in a view herewith, for erecting the bridge spans, in order to do away with all erection falsework. We have the following descriptive data from the contractor:



SPECIAL DERRICK-CAR SETTING 100-FT. PLATE GIRDER IN MEADOW VALLEY, WASH., RECONSTRUCTION OF SAN PEDRO, LOS ANGELES & SALT LAKE R. R.
(Built and operated by Shattuck-Edinger Co.)

The spans to be erected comprised two 90-ft. and nineteen 100-ft. plate-girders, and ten through-truss spans 110 to 150 ft. long. The contract for the erection of the spans requires that running falsework and the track thereon be placed by the railroad company in each crossing, on which to operate derricks and travelers, prior to the erection of the steel structures. However, in view of the number of girders to be placed, and as falsework was not necessary for operation of traffic, it was recognized that a method of setting the girders without the use of falsework was very desirable, resulting in a saving for this item alone of about $200 per span. To this end the contractor's engineer, Mr. F. S. Edinger, designed the car-derrick shown in the photograph herewith, by means of which 100-ft. plate-girders for either through or deck spans, weighing 36 tons, are picked up at the center, taken to the site and lowered to the bridge seat.

Owing to the gage of the track on which the derrick moves being only 4 ft. 8½ ins. (standard), and the load being suspended 60 ft. beyond the forward truck of the car, it was decided not to attempt to swing the girders out from the center line of track to their final position, as the center of gravity would be outside the wheel-base, and a swinging boom of the length in question is difficult to control. The car therefore was built with fixed central boom; it sets the bridge girders on rails placed transversely on the abutments and piers, and they are skidded out to correct position afterward.

The derrick body is a flat-car of 100,000 lbs. capacity. At the forward end is a wooden A-frame, a 56-ft. central boom projecting forward, and a 20-ton swinging boom having a reach of 10 ft. Two longitudinal sills on the deck of the car, spaced 8 ft. 9 ins. out to out, carry the derrick framing and machinery, and they also project back of the car 18 ft. over a second flat-car which is used as tail counterweight when required. A toggle hanger at the rear end of the sills may be operated to take the weight of the rear car and its load, when the derrick is lifting a girder. At other times the toggle is released and the rear car is free to move under the projecting sills.

The boom is made of two 14 × 14-in. × 56-ft. sticks of selected Douglas fir, and is triangular in plan, being spread at the heel 8 ft. 9 ins. out to out, the same width as the derrick sills, against which it abuts. The A-frame legs are extended outside the car below the deck, so that blocking can be placed under them to provide lateral stability when the auxiliary boom lifts a load at the side of the car. This swinging boom is used in connection with another derrick for unloading girders from cars and for handling floor members of through spans and material for truss spans.

Back of the A-frame is a folding tower. This is used for erecting truss spans. With the tower folded down the car can pass through tunnels and bridges.

A Civil Service Examination for Deputy Commissioner of Public Works, Evanston, Ill.

[Contributed.]

The questions below were given at an examination held on May 1. This examination was open only to residents of the State. The salary of the Deputy Commissioner is to be $125 per month, and the qualifications were three years of practical work and technical engineering knowledge equal to that required for a bachelor's degree. No candidate was successful at this examination, and another, open to non-residents also, was held on July 3.

It is the personal opinion of the contributor that someone worth more than the salary offered will be required, if the questions are any criterion. He also calls attention to the formula here called Kutter's, which he has known as Chezy's, and to the value given for the coefficient C and the question about the modification for tile and brick sewers.

The Deputy Commissioner will be chief assistant to the Commissioner of Public Works. The scope of the Deputy's work is suggested by the duties required of the Commissioner. Briefly stated, the duty of the Commissioner is to perform all such services in the prosecution of public improvements as require the skill and experience of a civil engineer, together with such other services as are required of him by ordinances and resolutions of the City Council. His chief work is to plan and direct the construction, operation and maintenance work incidental to the streets, alleys, sidewalks, water mains, pumping station, sewers, etc.; to approve subdivision plats and building plans submitted for permits, and to issue permits for and control the work done, in streets and alleys, by various public utility corporations.

It is believed the questions are so prepared that the use of engineering manuals will not be necessary. It is suggested that applicants answer those questions found least difficult first. Proper final numbering of sheets will bring replies in the same order as questions. Applicants will designate questions not answered by their number and writing "Not answered."

QUESTIONS.

(1) State briefly what practical or educational training you have that, in your judgment, may fit you for the duties of the position sought to be filled by this examination.

(2) State briefly what, if any, important construction or investigational work, public or private, you have been connected with as engineer, inspector, superintendent or foreman. In answering, state your rank and length of service and approximate number of men that were subject to your direction.

(3) Draw to a scale of 880 ft. to the inch a diagram representing a section of land and designate thereon, by shading, the tract legally described as the W. ½ of the N. E. ¼ and the N. E. ¼ of the N. W. ¼. How many acres is represented by the unshaded portion of the diagram?

(4) The roadway of Davis St., across Ridge Ave., is macadam. How would you prepare the same for receiving a single course of paving brick, using a sand cushion and sand filler? How many paving brick and how much sand, would you need to complete the work? You may assume the roadway of Davis St. at 36 ft., and the width of Ridge Ave. at 80 ft. Show your work in full, and state assumptions made to answer the question.

(5) State briefly the essential requisites and tests you would employ to determine the quality of the following materials, to wit.:

(a) Paving brick.
(b) Vitrified tile sewer pipe.
(c) Cast-iron water pipe.
(d) Portland cement.

(6) State briefly the theory of action of septic tanks, intermittent sand filters and sprinkler filters in the purification of sewage. Explain the bacterial action and the function of "grit" and "dosing" chambers.

(7) State briefly the main differences between:

(a) Fire tube and water tube boilers for water-works.
(b) Vertical and horizontal pumping engines for water-works.
(c) Centrifugal and screw pumps for sewage pumping.
(d) Feed pump and injector for forcing feed water into a boiler.

(8) How does the steam pressure in a boiler affect the boiling point of the water in a boiler?

(9) Given, an Illinois coal having a heat value of 12,500 B.T.U., and knowing the mechanical equivalent of 1 B.T.U. is one Joule, or 778 ft. lbs.,
Find how many pounds of such coal will be required to be burned to run a 50-HP. engine 10 hrs., provided 1-6 of the total heat in the coal is used in doing the work.

(10) Given the latent heat of water as 144 B.T.U. and the latent heat of steam at 966 B.T.U.,
Find how many pounds of ice at 32° F. can be melted by 3 lbs. of steam at 212° F.

(11) (a) Solve $(a+b)^3$
Factor (a^2-b^2)
(b) Write the formula for finding the area of a circle whose radius is R.
Write formula for finding the area of an ellipse whose principal axes A and B are known.
(c) Draw a right-angle triangle and complete the formulas for the following trigonometrical functions of the angle A included between the base and the hypothenuse, to wit:
Sin A; Cos A; Tan A; Sec A.
Draw an oblique triangle and show derivation of the equation for finding unknown sides when two angles and one side are given.
(d) Derive the equation of a circle referred to rectangular axes X and Y with origin at center of the circle.
(e) Write the differential of x^3.
(f) Write the integral of $5x^4\,dx$.

(12) Stability against the overturning force of wind requires proper design and proportioning between weight, height, breadth of base and exposed area of a smoke stack, tower, etc. This relation is expressed by the equation

$$W = \frac{d\,h^2 C}{b}$$

C = 56 for square shape, 35 for octagon and 28 for round shape.
d = average breadth of shaft in feet.
h = height of shaft in feet.
b = breadth of base in feet.
W = weight of shaft in pounds.

(a) Find the necessary weight of a square shaped chimney 100 ft. high with base 9 ft. wide and top 5 ft. wide to be safe against overturning.
(b) Assuming brick masonry weighs 110 lbs. per cu. ft., find the average thickness of the walls required.
(c) State, by inspection of formula, what reduction in weight could be made by substituting a round shaped chimney if the average breadth of shaft and breadth of base remained unchanged.

(13) Employing Kutter's formula, $V = C\sqrt{RS}$:
(a) Assume that the value of C as given in hydraulic tables for such conditions is 116; compute the delivery in gals. per day of a 24-in. water main 10,000 ft. long, under a drop in pressure of 0.868 lbs
(b) In what particular would the equation or its factors be modified to solve the problem for the discharge of a tile pipe or brick sewer of equal size or length under a drop of 8 ft.

(14) What force or "pull" parallel to the roadway must be exerted to sustain a wagon and its load upon a roadway having a grade of 4 ft. per 100 ft. In solving disregard friction and determine the theoretical force only.

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The Original Proposal for a High Building for New York Municipal Headquarters.

Twenty-four years ago, the late Arthur M. Wellington, then Editor-in-Chief of this journal, proposed a "loop" plan for the Brooklyn Bridge terminal in New York City, and suggested as one feature of the plan the construction over the terminal station of a high building to be used for municipal offices.

Mr. Wellington as an engineer was always phenomenally sanguine in his forecast of future development, but even his anticipations have been far surpassed by the enormous development of the high office building. It is of interest to note that the new municipal building, which is under construction by New York City, and is illustrated in this issue, while it does not stand on the site of the present Brooklyn Bridge terminal station which Mr. Wellington proposed to remodel, stands on a site within 200 ft. of it which was acquired by the city on which to construct a loop subway connecting the Brooklyn and Manhattan bridges. In a measure, therefore, it realizes and goes far beyond Mr. Wellington's project of 24 years ago, which was at that time thought by many to be quixotic.

It is also of interest to note that a loop terminal with a great office building over it has been in operation for two years at the Hudson Terminal, only a few blocks away, where the tunnel lines under the North River have their downtown station.

An Exposition of the Advantages of Direct City Construction Instead of Contract Work.

One of the questions always open to discussion is whether or not it is advisable for a city to undertake the direct construction of public works instead of intrusting their execution to a contractor. The question recently arose at New Orleans and was made the subject of a strong and interesting report in favor of the city undertaking the construction of a drainage canal with its own forces. This report, which was written by Mr. George G. Earl, M. Am. Soc. C. E., is printed elsewhere in this issue.

As we have already intimated, Mr. Earl made out a strong case for doing the work by the city directly. Notwithstanding this the city did not undertake the construction, as we understand it, owing to legislative prohibition found by legal talent in either the city charter or in the statute authorizing the drainage work.

We leave to others to discuss, if they see fit, the arguments put forth by Mr. Earl, merely reasserting our opinion that they are strong arguments in favor of city construction in this particular case. What we wish to call especial attention to, in addition to directing our readers to Mr. Earl's report, is the unfortunate way in which nearly all our cities are hampered when it comes to making a choice in matters of public policy like the one under discussion. We think that few would contend that the choice between carrying out public works by contract or by direct city labor involves any grave fundamental principles which must be established by the state. It may be that the legislative requirement of contract work at New Orleans was more or less a matter of chance. Certainly it is very unlikely that members of the Legislature of Louisiana gravely debated whether or not the city of New Orleans should be allowed to construct public works direct or be compelled to do it by contract.

We believe that city authorities should be permitted a free choice between the two methods of building public works. It may be highly advantageous to have the power to execute public works by day labor forces even though the power be seldom exercised.

Selling Engineering Work at Auction.

We commented in our issue of July 6 on the advertisement of the city of Carbondale, Pa., for sealed proposals from "consulting engineers" to prepare plans for "a comprehensive system of sewerage and sewage disposal." Bidders were required to submit cash or certified check for $50 with their bids, to be forfeited if they failed to enter into contract within 10 days from the award and to complete the job within 60 days. Further, no money was to be paid the engineer for his plans unless and until they were approved by the Pennsylvania Commissioner of Health.

It may be of interest to our readers to learn what bids were received when they were opened on July 10. The list was as follows (the names of the bidders being omitted):

(A) Complete design, including all necessary surveys, maps, plans, etc.; $1,250.00 plus traveling expenses, or $25.00 per day and $10.00 for assistant.
(B) Complete design, plans and specifications; $800.00. For surveys, maps, etc., $20.00 per day.
(C) Complete design, plans and specifications, $500.00.
(D) Complete design, including all necessary preliminary surveys, report, maps, plans, specifications, etc.; $780.00.
(E) Complete design, plans and specifications, no surveys; $539.00.
(F) Complete design, as per advertisement; $972.00.
(G) Complete design, including surveys; $1,175.00. If City Enginer make surveys deduct $225.00.
(H) Complete design, maps, plans, specifications, reports, surveys, etc.; $900.00.

Note.—All the bidders, except (D) and (E), enclosed a certified check for $50 with their bid. (E) wrote a very dignified letter to the members of the council to the effect that their request for a certified check was unprofessional. The contract was awarded to (D).

One of the bidders has written us as follows:

Sir: Having read the editorial in your issue of the 6th inst. criticising the City of Carbondale, Pa., for its action in advertising and requiring certified checks accompanying sealed proposals of engineers who desired to compete for the making of plans and report on a comprehensive system of sewerage and sewage disposal works, I feel some comment on the same is desirable.

In the first place, the result of the bidding was exactly as you anticipated: the successful bidder is one whose engineering training was obtained in a rolling mill in great part, and whose engineering practice has been largely the sale of vacuum sweepers, although recently given to the guaranteeing of odorless sewage purification without any attention to the apparatus on the part of an operator, however irregular the influent.

The facts in the whole matter indicate, however, that there were no "interests of the engineering profession" there to protect,—at least none such as inspired the editorial. The laws of Pennsylvania, based on the European practice commented on in your editorial of April 13, 1911, require that such work shall be advertised and let by competition to the lowest responsible bidder; but, having in mind the "unprofessional" aspect of the matter, the city had on a previous occasion obtained private proposals from a number of engineers, who generally offered to do certain stipulated things for a stated consideration, ranging all the way from $400 to $2,500. Inquiry developed that a number of the engineers were interested in the introduction of patented apparatus; others had little experience in the design of public works; and there was one instance of a prominent firm which repudiated its own plans for another municipality because of a fancy for apparatus more recently come into favor with that firm. To avoid criticism, no one was selected for the work at that time.

The public letting was attended, as was expected, by representatives of a number of previous solicitors for the work, including several low bidders interested in the sale of patented apparatus and whose names appear frequently as having been "selected" by municipalities, as well as reputable and capable engineers who proposed to render complete and satisfactory service at prices within range of choosing. There were a few absentees of previous solicitors, whose efforts, however, would have received the same reward imposed by the Pennsylvania law as that received by the high bidders.

From all this it is evident that so long as there are pretending engineers, sales engineers disguised as members of the American Society of Civil Engineers, and engineers who incur big bills on their reputations, as well as conscientious engineers who respond to calls for service from communities which are concerned but once in a lifetime in the selection of a capable engineer, there is no engineering profession which has any interests that are not constantly open to the commercialism of the times, and the council chambers of our smaller cities are no places to be delicate about such supposed interests when the reputations of the members of councils depend upon actions above criticism.

Respectfully,

Bidder.

July 17, 1911.

If, as our correspondent states, the laws of Pennsylvania require municipal engineering work to be advertised and let to the lowest responsible bidder, it is time the engineering societies of the state exerted themselves to have the laws altered. How long would the State Medical Societies rest content under a law which required physicians in public hospitals to bid for their places? Suppose the district attorneyship throughout the state were to be put up at auction. Would the legal profession stand for it, or the public? Except for the fact that engineers, taking them as they come, will average up to a higher scale of honesty than the lawyers, the auctioning off of public engineering work is just as injurious to the public welfare.

But the question is, How can the average city council know how to select a competent engineer? Where can they obtain any authoritative, unbiased advice? The lack of an answer to this question is costing the engineering profession dear.

The Design of Reinforced-Concrete Standpipes.

The paper on the design of reinforced-concrete tanks and standpipes recently read before the Boston Society of Civil Engineers and the consequent discussion, both of which are abstracted on another page of this issue, make an important addition to the general knowledge of this comparatively recent type of construction. The substitution of concrete for the widely used steel tank has been generally made as a matter of economy, both in first cost, replacement charges and maintenance. Secondarily, it was hoped that a construction had been found more pleasing to the eye than the ordinary steel standpipe or tank. As a rule the first expectation has been fulfilled; but except where a considerable sum has been spent in architectural detail,

the concrete standpipe has not usually added beauty to the landscape.

In some few cases, mainly in Europe, considerable money has been spent to give a pleasing form to concrete tanks and in at least one instance, a facing of cut stone has added measurably to the appearance, but the usual design is a cylindrical shaft, sometimes of changing diameter, but more often a straight cylinder, with a smooth concrete surface in practically every case marred and splotched with moist spots or efflorescent salts.

It is these disfiguring patches that take away any artistic value a plain concrete shaft might have and, what is more, arouse in the laity apprehension as to the watertightness and consequently the safety of the structure.

That this apprehension is unfounded is shown by the record of the concrete tanks which have been built. Out of fifty odd that are on record, only one has failed, and that structure was palpably of unsafe design. Nevertheless, since even minor leaks are undesirable and efflorescent splotches are displeasing, every effort should be made to procure a wall impervious alike to percolation of water and the leaching of salts. It has been generally admitted that the primary precaution necessary is a good concrete, rich in cement, with well graded aggregate thoroughly mixed and deposited; but Mr. Andrews, in the paper under discussion, proposes a change in the theory of design as well. He has applied his method with some success in two recent standpipes and recommends it for future use.

Up to some ten years ago the engineering world accepted, with some doubts, the anomalous theory of Considère which held that concrete surrounding steel, so that the two materials act together, has a much higher modulus of elasticity under tension than has plain concrete. Later experiments, however, showed that when reinforced concrete is put under tension, the elastic stretch in the concrete continues only until it reaches its ultimate tensile value, which is the same in both plain and reinforced concrete, and that above that point (when the steel is stressed much lower than its safety limit) minute cracks form in the concrete destroying its tensile value, and throwing all further elongation into the steel. These cracks are microscopic in character up to the highest working values in the steel, and while causing the rejection of the tensile value of the concrete in any reinforced concrete design, have never been considered a serious detriment to safe construction. In fact, tests by Prof. Probst in Germany have shown that the tension cracks which occur in the concrete when the steel is stressed below its elastic limit, are not of sufficient size to permit the entrance of a corrosive gas in sufficient quantity to cause rusting in three days, though exposed steel rusted in the presence of the gas in 24 hours.

To avoid these minute cracks in reinforced-concrete tanks is the intention of Mr. Andrews' new method of design. Hitherto it has been the practice to design the horizontal steel in reinforced-concrete tanks of sufficient area to take all of the tensile circumferential stress due to the water pressure and to encase that steel in a concrete wall of an indeterminate thickness, generally regulated by considerations of construction. Mr. Andrews thinks that these walls have been too thin and in consequence when the full water pressure has prevailed, the steel has been subjected to its designed unit stress of 12,500 or 16,000 lbs. per sq. in., as the case may be, the composite material has stretched beyond the tensile value of the concrete and the typical minute cracks have appeared. Through these cracks, invisible to the eye, the water has passed in greater or less quantity, forming either leaks or efflorescent splotches.

To remedy this, Mr. Andrews proposes to design the concrete-steel walls of sufficient thickness so that the concrete may never be stressed beyond its tensile strength. In doing this the steel, of course, carries only a small part of its safe load, but the concrete wall remains intact and, provided the material is good and well placed, impervious.

This theory may account for the general areas of moisture and of efflorescence which disfigure the bases of standpipes. It is quite possible that the vertical cracks are so numerous and so closely placed that whatever of moisture or salts passes through them seems to be fully distributed over the face of the tank. At Attleboro, for instance, the entire bottom outside face for a height of some 10 ft. above the ground was one mass of caked efflorescent salt, when seen by a member of our staff, about two years ago. This porous percolation may have been due to a uniform lack of density in the concrete, but it could also have passed through the tensile cracks undoubtedly formed by the pressure of 100 ft. of water in the tank.

On the other hand, leaks passing appreciable amounts of water are so unevenly distributed over most of the leaky tanks that it seems improbable that the minute-crack theory will explain them. The chances are that these leaks are due to some defect either in the material or the manner of placing it. The very general efflorescent streaks along the horizontal line of the end of the day's work are obviously the result of an imperfect bond of the new and old concrete. The new method of design will not be a sovereign remedy to cure leakage troubles, but the additional cost of the extra concrete required by it is low enough to justify its use, provided it proves effective in practice.

Mr. Andrews' method of obtaining the ultimate tensile value of the concrete in a tank wall does not seem to be warranted by present knowledge of the behavior of concrete. He had tested in tension a number of 4-in. section pieces and to obtain a probable tensile fracture limit for concrete in place in a wall, he first adds 25% to the test figure to allow for expected increase in a larger size piece and then 10% to allow for the additional tensile strength of concrete when reinforced. We know of no extensive tests that justify either of these increases. Tests to determine the effect of size of specimen on strength of concrete are extremely limited and, such as there are, are very unbalanced, and the best tests on reinforced concrete in tension show that the minute cracks begin to appear at about the tensile limit of the concrete as determined by tests on plain concrete of the same age and mixture.

More Light on the Forest and Stream Flow Controversy.

The long and violent controversy concerning the influence which forests exert on the flow of streams has been carried on to a large extent with isolated examples observed casually by individuals as the basis of argument. Those who wish to get at the truth in this matter will study with interest the results of two important investigations which have just been made public and which were made to determine if possible the effect of forests on stream flow.

The first of these was made by Col. Edward Burr, of the Corps of Engineers, to determine whether any changes in the flow of the Merrimac River in New England could be traced to changes in the forest cover on its watershed. The second was made by Prof. Daniel W. Mead, of the University of Wisconsin, to determine what influence the cutting off of the forests which formerly covered a large portion of Wisconsin has had upon the flow of Wisconsin rivers.

We give much space in this issue to an abstract of Col. Burr's report. Prof. Mead's report was reviewed in our Engineering Literature Supplement of July 13. Both these investigators reach substantially the same conclusions; to wit: That forests, so far as their investigations can determine, have practically no effect whatever upon the flow of streams.

So far as the influence of forests upon flood heights was concerned, this conclusion agrees, we are sure, with the opinion of almost every engineer who has given any careful study to the question. In the editorial page of this journal for June 27, 1907, years before the controversy over forests and stream flow assumed an acute stage, we said:

> A forest cover has comparatively little effect in reducing the height of floods. Instances without number are on record where most destructive floods have occurred in regions still covered with primeval forests.

Both Col. Burr's investigation and Prof. Mead's bring out very clearly that the overwhelming influence in causing floods, on the one hand, and low water, on the other, is a surplus or deficiency in rainfall. To cite examples of very recent times, we may take the floods which prevailed through Pennsylvania at the time of the Johnstown dam disaster, or those which caused destruction in the valleys of the Delaware and Passaic Rivers seven or eight years ago as a result of a phenomenal rainstorm. It needs only common sense for one to understand that such rainfalls would have caused disastrous floods even though the areas on which they fell had all been clothed with primeval forests.

When we go to the other extreme and consider the comparative maintenance of low water flow from forested and cleared areas, we are admittedly on more debatable ground. There is undoubtedly a wide general belief not only in the minds of the lay public but among engineers that brooks and streams flowing in the forests do not dry up in times of drought as do the streams flowing from cleared areas. On the other hand, it is evident enough that here again the influence of the forests is a very small factor compared with the lack of rainfall. It is well known that in a long-continued period of drought the streams flowing from a forested region will shrink away to mere threads. In fact, the volume of flow of streams at such times depends far more on the contributions they receive from springs and similar underground sources than upon the cover or lack of cover upon their watershed.

It thus appears that the variation in the flow of streams is first of all dependent upon the rainfall and almost wholly on the rainfall, so that the influence of forests to either increase the low water flow or diminish the volume of floods would be in any event but a small percentage of the total stream volume. If, now, the rainfall were uniform year by year and we had available accurate data as to the forest conditions on a given watershed at different times, we might proceed to investigate with considerable accuracy and settle definitely what this small percentage of forest benefit, provided it existed, amounted to.

But the rainfall is subject to the widest variations not only from year to year but from decade to decade. We are now passing through, for example, a period of deficient rainfall, the results of which are giving anxiety to water-supply engineers over a wide area. A few years ago, on the other hand, we were passing through a period of excessive rainfall, when engineers were experiencing anxiety lest reservoir walls and waterpower dams should yield under the stress of heavy floods. The difficult task of the investigators is to take the stream flow records, subject as they are to extreme variations from unknown causes, and determine whether a connection can be traced between any of the variations and the cutting off of forests or the regrowth of timber on lands once cleared.

It is at this point, if any, that the conclusions reached by Col. Burr and Prof. Mead will be assailed by the forestry enthusiasts. Each of them has done the best he could with the information available, but there is no very reliable information as to the changes which have taken place during the last half century with respect to forests, either in New England or Wisconsin. We know, indeed, that the valuable timber has been removed from vast areas, but to what extent a second young forest is taking its place and to what extent the land is cultivated or used as pasture there is no data available which is accurate enough to be relied upon. The published figures giving the area of forested land, as determined by census reports, etc., when analyzed and studied as to their origin, will be found to be little more than a summation of the guesses of farmers and woodsmen.

It is possible, of course, to study very small streams and compare known forest conditions on their watershed at a former period with pres-

ent conditions. Knowledge of changes in some such small streams is the basis on which much of the argument in favor of forests as conservators of stream flow has been founded, but it is apparent that what applies to a small brook may not hold good at all for a large river. The flow in the lower Merrimac, for instance, in a time of drought may be benefited as much from a local thunder shower falling on some area in its watershed where the forest has been removed and a large part of the water falling runs off, as it could be benefited if the same watershed were covered with trees.

It is noteworthy that the question of the relation of forests to stream flow is at the present day one on which the direct expenditure of millions of dollars is involved. Congress has made large appropriations for the purchase of forested lands in the mountains of New England and in the Southern Appalachians, but the United States Geological Survey has first to certify that the forests on these tracts of land are essential to benefit the navigation of the rivers which there have their sources. Where is the proof that the forests produce this beneficial effect? The investigations of Col. Burr and of Prof. Mead all go to show the contrary.

There is, on the other hand, more or less evidence to show that the changes which have taken place in the last three-quarters of a century have been injurious, to some at least, of the streams of New England and of the South, partly in the way of drying up of the low water flow and partly through the amount of sand and gravel carried down by the rivers in high floods.

It seems to us, however, that so far as these changes in stream flow and stream beds have occurred, they are traceable more to what has accompanied the removal of the forests than to the mere loss of the forest cover. For example, many rivers of New England in their primeval state were lined with swamps and morasses. With the settlement of the country these swamps were cleared and drained and became the fertile meadows of the present day, but the storage afforded by this broad area lining the streams was, without doubt, of importance in maintaining the low water flow. Again, it is not the mere cutting off of the forests that causes erosion of the hills. There may be no more erosion from a tract of underbrush or a well-grassed pasture than from a forest. It is where land is cultivated and the soil exposed to washing and where roads form a channel for surface water that erosion takes place, sending down great bodies of sediment to fill the valleys of the streams.

It looks very much as if in the case to establish the forest as the conservator of navigable waterways the Scotch verdict "not proven" will have to be rendered.

If any of our readers are inclined to question this, it will be well to reflect that if forests are to produce any real benefit to a navigable river they must increase its low water flow by a material percentage, or must actually and demonstrably reduce the amount of sediment brought down into the navigable reaches from the headwaters, usually hundreds of miles away.

Even if it can be demonstrated that forests do have some small favorable influence upon the flow of small streams, their case as respects the navigable rivers will not be helped, and it is this case which must be proved before the U. S. Government can proceed to take possession of the forests which clothe the slopes of the White Mountains and the Southern Appalachians.

LETTERS TO THE EDITOR

Lieut.-Col. Burr's Report on Forests and Run-Off, Merrimac River Basin, and the Almost Overlooked Low Rainfall at Lowell, Mass., 1831-45.

Sir: Permit me most earnestly to call your attention to the report recently printed as House Document No. 9 of the 62d Congress by Lieutenant-Col. Edward Burr of the Corps of Engineers, relative to the influence of forestry upon runoff of rivers. [The report is abstracted at length elsewhere in this issue.—Ed.]

You will find that this very careful study by Lieut.-Col. Burr and his assistant, Mr. Fred. O. Stevens, presents what are undoubtedly the best data yet available upon this continent relative to the influence of forests upon river flow.

It is naturally gratifying to me to find the views which I expressed in my address at the conservation meeting in the Engineering Building, New York City, two years ago, thus fully confirmed by this very careful study, but it is not altogether strange that I had an inkling of the truth, considering that I passed my first ten years after graduation with the water-power company at Lawrence and did much of the detailed work of computing the runoff of the Merrimac from the early records up to that time.

My Hudson River studies of three and four years ago all pointed to the same conclusion.

On the Genesee watershed, in very different topographic conditions, it is entirely possible that different results have obtained.

But the feature of most interest in this report which I wish to call to your attention is one which Mr. Hiram F. Mills had pointed out to me some years ago, namely, that the great drought or shortage of rainfall from 1880 to 1883, which most of our text books and most of our engineering reports on matters hereabout have recorded as the worst on record, was by no means so severe as that shown for the Lowell rain-gagings for a period of more than ten years between 1831 and 1845.

You will see that these are set forth on Diagram No. 6 [Fig. 3 in our abstract.—Ed.] of Lieut.-Col Burr's report, and you will note also how this shortage is confirmed by the rainfall records at Providence, R. I. The precipitation at Boston, Mass., as set forth in Sheet No. 7 [our Fig. 4] of Lieut.-Col. Burr's report, was not so much below the normal. A study of these three records day by day would be interesting. If I remember aright, the records kept at the cotton mill at Waltham also extended back to an early date, but perhaps not quite so far back as those at the Merrimac Manufacturing Co.

The matter to which I would call your attention particularly at the present time is that we have historic proof of a much more severe shortage of rainfall and for a longer period than that which is commonly accepted hereabouts as the most severe on record.

Very truly yours,

John R. Freeman.

Providence, R. I., May 22, 1911.

[We have held Mr. Freeman's letter until we could present in the same issue a lengthy abstract of the report which he mentions, including the two diagrams mentioned by him. Comment on some phases of the report will be found in our editorial columns this week.—Ed.]

A Voice From the Wilds of New Jersey.

Sir: A purposeful and apparently scientific article recently printed made a statement evincing more hope and courage than accurate truthfulness:

We have conquered the mosquito; now let us conquer the fly.

The first part of that sentence ought to have a leading place in every newspaper, with flaring headlines. It is news par excellence; who has known heretofore that "we have conquered the mosquito"?

By the limited experience of the writer, Perry's famous dispatch seems better suited: "We have met the enemy and they are ours." For not only have we met them but we still meet them daily. And they are ours; that is, they are our daily and nightly house companions; we give them food as well as shelter.

It is true, literally, that we have conquered the mosquito. The writer conquers an average of one every two minutes at this season. He hears tales that the genius of Col. Gorgas has actually conquered the Panama mosquitoes. Would that we had the Colonel in New Jersey—with Uncle Sam's millions to back him!

It is heroic of course to see victory far in advance, and proclaim it from the house tops. Active members of boxing clubs are known to say before the handshake, "I've got him licked." It is the same with the boast: "we have conquered the mosquito." Scientists, campaigns of extermination, entomology treatises and other weapons have been put into the arena against the mosquito, and the dreamer of visions, fired with the spirit, says "We have conquered the mosquito." But we keep on slapping just the same.

This is not engineering; and may not deserve a place in Engineering News, but it concerns a good many engineers; and some headlights of the profession have won fame as mosquito exterminators. Oh, that they would all live up to their fame and really conquer the mosquitoes—or at least tell us why their efforts have come to naught.

"Bitten."

Singhurst, N. J., July 19, 1911.

The Late Howard Schuyler's Work on the Mexican Central R. R. Location.

Sir: Referring to your editorial in Engineering News of July 20, "Wanted, More Records of Pioneer Engineers," I am sure that the great majority of your readers are as much interested in the human side of the engineer's work as the scientific side.

I had the good fortune to be associated with Howard Schuyler while he was Chief Engineer of the Mexican Central R. R., and the southern end of this railroad is a monument to his ability as a locating and constructing engineer. His early death cut short a career which would have made him more widely known and honored, although he had then a national reputation.

Yours truly,

Emile Low, M. Am. Soc. C. E.

606 Ashland Ave., Buffalo, N. Y., July 22, 1911.

The Hamburg Gas-Holder Failure.

Sir: Referring to the account, in your issue of July 6, 1911, of the failure of a post in a gas-holder in Hamburg, Germany, which wrecked the gas-holder itself, it seems to me that it is time engineers should know something of the strength of a slender column and should recognize a slender column when they see one. So high an authority as Prof. Müller-Breslau should be able to offer to the engineering profession something more substantial and conclusive than the statement that "the use of tie-plated columns, when the section is assumed to be integral, may lead to constructions which do not afford adequate security under loading of unusual character."

At this long range, and with rather incomplete data, I would offer the conclusion that the column which failed had an ultimate strength somewhere between 66,000 lbs. and 263,000 lbs., with the odds in favor of the lower value. This is based on a theory which every engineer ought to know. It is said to have borne a load of 133,000 lbs. If the post had pin ends or their equivalent, 66,000 lbs. would have been the ultimate strength. If the ends were sufficiently rigid to make the points of inflexion seven-tenths of the length apart, the ultimate strength would have been about 263,000 lbs. Here is a very nice practical demonstration of theory, though a very serious experiment.

From the data of your excellent description I have constructed the sketch herewith, showing

the column in question. The dotted lines show how the column can bow under endwise compression. The tie plates do not hinder this bowing action except in an inconsiderable degree. Because of the absence of lattice these two channels can bow together, and the ratio of slenderness of the column is the full length of the column divided by the least radius of gyration of a single channel, or in this case 180.

I have many times condemned so-called columns just like this one. More than once I have suggested to investigators that they test members of this sort in compression. The head of the engineering department of a university asked for ideas for student investigation. I advised him to make tests of such members. He probably took the advice—and filed it.

It is a serious error to use Euler's formula for a short column, say where the ratio is below about 100, as was done in connection with this column. It is also a serious error to consider a column such as this one, which has practically no ability to carry transverse shear, as an integral column.

The uncertain feature of this column, which makes it impossible to determine, from the data given, whether the capacity is nearer to 66,000 lbs. or to 263,000 lbs. ultimate strength, is the unknown condition at the ends. If each channel is connected, as indicated, with two rivets, the degree of fixed-endedness is small, and the ultimate strength is correspondingly remote from 263,000 lbs. The column as a whole may be far from being fixed-ended. A large number of columns are designed as square or fixed-ended

which are in reality nearer to a condition of being pin-ended. There are but few cases in structures where members are truly fixed-ended or even approximately so. A column resting square on a masonry support is one of these cases, for that end at least. A light compression member riveted solidly or bearing squarely against a member very much heavier and stiffer than itself is practically fixed-ended. Two equally strong (or equally weak) compression members butting squarely against one another are not fixed-ended, for the tangent line drawn to the axis of each may deflect simultaneously, whereas in a fixed-ended member this tangent must be perfectly rigid.

There are many things that ought to be written into the literature of engineering that would go a long ways toward the conservation of life and property. If a lot of intricate mathematical rot could be eliminated and such common-sense truths as these substituted, engineers would learn how to design safely.

Yours very truly,
Edward Godfrey.

Monongahela Bank Bldg., Pittsburg, Pa., July 13, 1911.

Kutter's Formula Values for Rock Channels.

Sir: In reply to your correspondent's enquiry in your issue of April 20, respecting the value of n in Kutter's formula, the coefficient of roughness, I have the following experience to offer. A canal (irrigation) in which the water was flowing with a mean area of cross-section 12 sq. ft.; the cutting was through rock as per sketch herewith. I measured the velocity by float, taking the mean of several trials.

I checked this by another method known as the measurement of the ripple in inches and this coincided with the float velocity. The velocity was $1.22 \times .8 = .976$ ft. per sec. Taking Flynn's Kutter $n = .02$ gave me .49 more than the float, while $n = 0.0225$ gave .084 less than the float, so that in selecting the coefficient of roughness I adopt the mean of 0.02 and 0.0225 for an open rock canal somewhat rough at the sides, smooth at the bottom and no weeds or other obstruction. This again checks with Flynn's Kutter description of $n = .020$ for "rubble in cement in an inferior condition, coarse rubble set dry"; and $n = .0225$ for "canals in earth above the average in order and regimen."

I think 0.035 is too much, and would be only necessary in case of neglect, through weed growth, fallen rocks, stones, etc., which is not likely to happen on a well regulated water course.

J. De Lacy,
Water Supply and Irrigation Engineer.
P. O. Box 1656, Cape Town, South Africa, May 25, 1911.

Sir: Referring to the inquiry of Mr. Mugnier in your issue of April 20, regarding experimental data on the Kutter formula value of n for rock-cut channels, I can say that on the Chicago Main Drainage Canal, in rock section, values of n have been observed between 0.024 and 0.029. The latter is probably the more accurate.

The channel is 160 ft. wide and 23 to 24 ft. deep. The sides are vertical, channeled smooth in limestone rock; bottom rough.

In the experiments Q varied from 6,000 cu. ft. per sec. to 8,000 cu. ft. per sec., and S from 0.000,01 to 0.000,02. This channel may be outside of the limiting range for a constant value of n in the Kutter formula.

L. K. Sherman.

Chicago, Ill., June 9, 1911.

Brittleness of Metals Caused By Corrosion.

Sir: Having read in Engineering News, June 29, 1911, the note on "Brittleness of Metals Caused by Corrosion," and having noted the reference to a failure of Muntz metal due to "dezincification," I am reminded of a case of such action in sheet-brass that came to my notice many years ago. Many thousands of "cannon primers" had been stored in an old building at the Watervliet Arsenal and, the dampness being considerable, the nitre of the rifle powder which filled the small brass tubes had completely oxidized the zinc, leaving the copper spongy and brittle. White patches of zinc oxide covered the outside of the brass and, when broken, the metal showed the red copper only. So weakened were the brass tubes by such corrosion that it took less force to break them across than it did to pull out the friction-wire at the top, so that when the gunner jerked his lanyard he "spiked" the gun instead of discharging it.

Yours truly,
W. P. Mason.
Rensselaer Polytechnic Institute, Troy, N. Y., July 19, 1911.

Notes and Queries.

Mr. A. F. Harley, 136 East Bay St., Jacksonville, Fla., sends us a photograph of a map showing a survey of the Lakewood Estates, Orange Co., Fla., divided into lots for high-class residences and 5-acre tracts for orange groves and truck farms. The tract includes four small lakes, and boulevards have been laid out encircling the borders of the lakes, with curved streets running parallel to them and a tier of lots between. The map is a suggestive illustration of the manner in which such a tract can be laid out with far better esthetic effect than by following the ordinary plan of cutting up into rectangular blocks.

Engineers interested can probably obtain a copy of the photograph by addressing Mr. Harley.

ESTIMATING FLOOR ARCHES.—Information is asked by a reader as to the current method of estimating tile floor arches in square feet. We understand the point of doubt to be whether the superficial area of floor between beam centers is taken, or whether an allowance is made over this to take account of the special skewbacks; also, whether the beam protection is taken to be included in the floor-arch estimate or is to be estimated separately. He says that he has found trouble in checking bills for fireproofing companies by the square feet of floor arches.

PULL OF FLAGS ON FLAG-POLES.—W. P. H., Milwaukee, Wis., asks whether we know of data on the pull of flags on flag-poles at different wind velocities. We do not. Information on the question would doubtless be of interest to others besides W. P. H. and ourselves.

DEPRECIATION OF ELECTRICAL PROPERTIES.—The paragraph in our account of the recent convention of the American Institute of Electrical Engineers (Eng. News, July 6, p. 31) describing the paper on "Depreciation" by Mr. Henry Floy, of New York City, failed to give an adequate idea of the scope of that paper, as the result of pressure of convention reports on space and time. The paper included, besides the items of theoretical and actual depreciation previously empnasized in Mr. Floy's discussion before the National Electric Light Association (erroneously referred to as a "paper"), a wide range of subjects such as (1) the necessity of uniform use of terms in discussing depreciation, (2) need of distinguishing rate of depreciation used for estimating from total sum of depreciation in physical property for a given time, (3) amortization of development costs, etc., (4) accurate determination of actual depreciation, (5) the need of broad cooperation for the collection of data on depreciation.

Progress Results of Exposure Tests of Paints for Steel, at Havre de Grace Bridge and at Atlantic City Panel-Test Fence.

Two important large-scale exposure tests of paints for steel, under the auspices of Committee D-1 (Preservative Coatings) of the American Society for Testing Materials, have been going on for five years and three years respectively. They are beginning to yield indications of value in judging of steel paints. We therefore reproduce here a summary of the most recent results, as reported by the above committee at the meeting of the society, June 27-July 1, 1911.

Havre de Grace Bridge Test.

Two spans of the Pennsylvania R. R. bridge over the Susquehanna River at Havre de Grace, Md., were painted in October, 1906, with 19 different paints, each on a separate panel length. The painting was supervised by the committee. At the same, for each panel nine steel plates 2×3 ft. were also painted with the corresponding paint, at three different spreading rates, and these were attached vertically on the outside of one truss, facing southeasterly. Periodical inspections of the condition of both bridge and plates have been made by sub-committees of Committee D-1.

The most recent inspection was made in April, 1911. On the day set for inspection of the bridge bad weather interfered and accurate ratings are therefore at hand for the plates only. The text of the report, however, contains remarks on the relative condition of bridge and plates.

The table of ratings given by the committee is reproduced here as Table I., omitting, however, the individual ratings of the six viewers and giving only their average. The report refers to the paints by panel number only. The paints were analyzed by members of the committee ("Proceedings" of the society, VIII, 1908). We have supplemented the table by a column giving the pigments of the several paints, after the table of analyses by Mr. P. C. McIlhiney.

REPORT ON HAVRE DE GRACE BRIDGE INSPECTION, APRIL, 1911.

The 1911 spring inspection of the Havre de Grace bridge was called for April 21 and 22, under the following program: The first day to be devoted to an examination of the plate panels, when it was to be determined which of these were to be removed for photographing before and after the removal of the coatings. The second day to be devoted to a thorough inspection of the bridge proper. Rain prevented the carrying out of the second day's program as originally planned. Plate examinations' schedules are as follows:

A.—Chalking.
B.—Checking, Cracking, Alligatoring, etc.
C.—General surface conditions,

with the following marking for each:

Excellent	10 to 8
Good	8 to 6
Fair	6 to 4
Poor	4 to 2
Failure	2 to 0

It is of record that the examinations showed oftentimes marked variations in the condition of the protective coverings under the different rates of application, and it may be assumed that this clearly proves that each paint has what may be styled its "natural covering rate," and that the panel rate approximating thereto generally showed the best. This in the majority of cases was the rate 600 sq. ft. per gal.

REVIEW.

Under A (chalking) the average rating of 14 paints was 8.7, Fairly Excellent; of 4 paints 7.4, Fairly Good; of 1 paint 4.5, Only Fair.

Under B (checking, etc.) the average rating of 16 paints was 9.7, Nearly Excellent; of 2 paints 6.6, more than Fair; of one paint 2.2, Failed.

Under C (general conditions) the average rating of 9 paints was 8.6, Very Good; of 8 paints 7.5, Nearly Good; of 1 paint 5.7, Very Fair; of 1 paint 2.3, Failed.

For comparison the average results under same scale of rating for 1910 report are listed. Only two of the individual inspectors this year are the same as reported last year, so that additional interest is given to this comparison of averages. Results generally show intelligent differentiation, with some deterioration in the vast majority of cases.

It was determined to remove the average general conditioned plate of the three panels at 900 sq. ft. per gal. rate, for each paint, for photographic examination, and this was done under the supervision of the Chairman of the Inspection Committee on May 2, at which time a thorough inspection of the bridge proper was made by W. A. Aiken, assisted by Mr. Anderson Polk. The panels were carefully packed and have been shipped to New York.

The result of the inspection of the bridge proper referred to above resulted in the following general conclusions, in practical agreement with that reported last year as the average of several individual conclusions, though from a much less detailed inspection of the bridge proper at that time.

It is undoubtedly a fact that generally the bridge structure is in as good and in the majority of cases better condition to-day than similar panels. While there may be shades of difference in the protection afforded, it would be difficult to draw these sharply at this time except in the cases mentioned below.

Paint 1, while meeting very fully the tests for chalking, cracking, etc., is of markedly poor character—in some respects more pronounced than any. What protection is being afforded, and this is better than on the panels, these averaging only Good, is being furnished by the under-coat.

Paint 2 is furnishing much better protection on the structure than on the panels, contrary to the previous indications of early disintegration as originally reported in 1908. This is undoubtedly due to the fact that the Superintendent of Tests on his original report at the time of undertaking these investigations marked the natural spreading rate of this paint to be 600 sq. ft., and it is this rate in panels which show best, though not as good as the bridge proper.

Paint 4 is exceptional in not furnishing as good protection to the bridge proper as on the panels.

TABLE I. CONDITION OF PAINT ON AUXILIARY TEST PLATES, HAVRE DE GRACE BRIDGE, AFTER 4½ YEARS' EXPOSURE.

Inspecting Committee: W. A. Aiken, M. McNaughton, A. H. Sabin, C. M. Chapman, A. Polk and J. H. Gibboney.

Individual ratings not reproduced. Rating given is average of the six individual ratings, each being an average for the nine plates comprising three different spreading rates.

Markings are for:
A — Chalking.
B — Checking, cracking, alligatoring, etc.
C — General surface conditions.
All on scale of 10 = perfect.

Paint.		1911 Rating	Inspection Remarks.	1910 Rating.	Nature of Paint Pigment* (after analyses by P. C. McIlhiney).
1	A	8.0	Very good	8.1	†Iron oxide 79%, red lead; iron oxide 54%, carbon, red lead.
	B	10.0	Excellent	10.0	
	C	7.0	Good	6.6	
2	A	8.2	Very good	8.5	Silica 41%, carbon, iron oxide; Carbon 41%, silica, iron oxide; Silica 53%, carbon, iron oxide.
	B	6.5	Fairly good	8.2	
	C	5.7	Fair	5.3	
3	A	7.6	Quite good	7.9	Carbon 96%; White lead 30%, zinc oxide, lead sulphate, clay; Gypsum 65%, iron oxide.
	B	9.8	Excellent	10.0	
	C	7.5	Quite good	7.6	
4	A	8.9	Fairly excellent	7.8	Carbon 88-92%, red lead. (All three coats).
	B	9.8	Excellent	9.8	
	C	7.6	Quite good	8.8	
5	A	8.5	Fairly excellent	7.6	Carbon 67-69%, red lead. (All three coats).
	B	10.0	Excellent	10.0	
	C	8.3	Very good	8.7	
6	A	8.6	Fairly excellent	9.0	Red lead 72%, silica; Carbon 21%, silica, red lead; Carbon 79%.
	B	10.0	Excellent	10.0	
	C	9.1	Fairly excellent	9.0	
7	A	8.2	Very good	8.8	Carbon 37-50%, lithopone, clay. (All three coats).
	B	9.8	Excellent	10.0	
	C	7.9	Very good	8.7	
8	A	9.1	Fairly excellent	9.2	Graphite 96-98%. (All three coats).
	B	9.8	Excellent	10.0	
	C	8.0	Very good	9.0	
9	A	7.8	Very good	9.2	Silicate 81-53%, graphite. (All three coats).
	B	9.8	Excellent	10.0	
	C	7.9	Very good	8.6	
10	A	7.3	Good	8.4	Red lead 100%. (All three coats).
	B	6.8	Excellent	8.0	
	C	8.9	Very good	9.2	
11	A	7.1	Good	8.2	Red lead 100%. (All three coats).
	B	8.3	Very good	8.4	
	C	8.7	Very good	8.8	
12	A	9.3	Fairly excellent	9.2	Silicate 47%, white lead, carbon, zinc oxide; Carbon 80%, red lead. (Last two coats).
	B	9.8	Excellent	10.0	
	C	8.7	Very good	9.4	
13	A	9.4	Fairly excellent	9.0	Clay 58-50%, graphite, iron oxide. (All three coats).
	B	9.8	Excellent	10.0	
	C	7.5	Quite good	8.1	
14	A	9.0	Fairly excellent	8.8	Red lead 80%, clay; Carbon 90%; Carbon 98%.
	B	9.8	Excellent	9.8	
	C	8.8	Very good	8.6	
15	A	4.5	Barely fair	3.6	Carbon 99%. (All three coats).
	B	2.2	Failed	3.5	
	C	2.3	Failed	3.6	
16	A	9.0	Fairly excellent	9.0	Carbon 48-55%, silicate. (All three coats).
	B	10.0	Excellent	10.0	
	C	8.4	Fairly excellent	9.2	
17	A	8.9	Fairly excellent	9.2	Iron oxide 48-50%, barytes, carbon. (All three coats).
	B	9.8	Excellent	8.8	
	C	7.6	Good	7.9	
18	A	9.0	Fairly excellent	9.4	Carbon 70-82%, lead sulphate. (All three coats).
	B	9.8	Excellent	9.9	
	C	8.7	Very good	9.2	
19	A	8.6	Very good	9.2	Carbon 90-93%, red lead. (All three coats.)
	B	9.6	Excellent	9.3	
	C	7.3	Good	8.8	

*This column does not appear in the report. It is compiled from a table of analyses in "Proceedings" VIII (1908). Arranged in following order: First coat; Second coat; Third coat. For each coat the ingredients of the pigment are given in quantitative order, and the amount of the largest is given. The original table of analyses does not always account for 100% of pigment.—Ed.
†This is a varnish paint. For panel 1 the tables of analyses account for two coats only.

Paint 10 on the bridge proper shows better protection than on the panels, even though these latter are rated Very Good. The paint, however, shows a marked tendency to decay, and is likely to fail in the near future.

Paint 14 on the bridge proper, while showing minute cracking of outer coat as noted three years ago, disclosing the under-coat of red lead prominently throughout the structure, indicates failure to be superficial, more apparent than real, and to-day affords good protection, better in fact than in many of the paints which lack any evidence of cracking, checking or alligatoring.

Paint 15 on the bridge proper, except in numerous well defined areas showing very badly throughout, is in better general condition than on the panels, which are all in very poor condition and are marked as having failed. The panels as a whole are typical in their condition of the well defined areas referred to above, scattered throughout the structure proper.

With the above special exceptions, all paints are affording excellent protection on the bridge proper. It was noted that generally on the horizontal struts and laterals and their connections, affording the easier accumulation of moisture, most of the paints show more or less deterioration. This only emphasizes the necessity for special care in painting structures. The light latticing of the main riveted members most markedly on the upstream side of the structure shows this tendency more or less in the majority of cases.

On May 25, in Brooklyn, N. Y., the photographing of the typical plates was begun in the presence of the Chairman of the Sub-Committee and A. H. Sabin. The removal of the coatings was subsequently accomplished. In all cases the plate material condition was found to fully corroborate the ratings given the several paints, as generally at this date affording excellent protection.

Atlantic City Fence Test.

The steel plates erected on a fence in Atlantic City, N. J., facing the ocean, were painted in October, 1908. They were inspected last on June 28, 1911, giving a record of the results of 2¾ years' exposure. The viewers were not the same as for the Havre de Grace bridge, and numbered four in place of six. Only a single average rating is given for each panel by each viewer. The pigment is stated in the committee's tabulation, which is reproduced here as Table II (again omitting the individual ratings). It is understood that those not specially marked are straight linseed oil paints.

The best ratings are summarized by the committee in a table reproduced here as Table III.

REPORT ON ATLANTIC CITY PAINT-TEST FENCE INSPECTION, JUNE, 1911.

On Wednesday, June 28, 1911, the second inspection of the Atlantic City Steel Test Panels (erected in October, 1908) was made by Sub-Committee D of Committee D-1, this committee having agreed to report upon the conditions of the painted surfaces, leaving any report on the comparative corrosion of the various types of metal used in the test to Committee A-5 on Corrosion of Iron.

According to the amount of rust apparent on the painted surfaces of the panels, as well as the degree of checking, chalking, scaling, cracking, peeling, loss of color and other signs of paint failure shown, ratings were given each panel. The system of rating which took into consideration all of the above conditions, was similar to the system used at the first inspection during 1910, when zero denoted the worst results and 10 the best results.

TABLE II.—CONDITION OF PAINT ON ATLANTIC CITY PAINT-TEST STEEL PANELS, AFTER 33 MONTHS' EXPOSURE. (JUNE, 1911.)

Inspecting Committee: H. A. Gardner, W. H. Walker, P. H. Walker, C. M. Chapman.
Individual ratings not reproduced. Rating given is average of the four individual ratings.
Markings are on scale of 10 = perfect.

Panel.	Pigment.	Average.
1	Dutch process white lead	3.7
2	Quick process white lead	4.2
3	Zinc oxide (xx)	1.5
4	Sublimed white lead	9.
5	Sublimed blue lead	8.8
6	Lithopone	2.2
7	Zinc white lead	4.7
9	Orange mineral	8.3
10	Red lead	8.3
12	Bright red oxide	8.1
14	Venetian red	8.
15	Prince metallic brown	6.3
16	Natural graphite	6.8
17	Artificial graphite	5.9
19	Lamp black	6.3
20	Willow charcoal	8.8
21	Carbon black	7.2
24	Yellow ochre (French)	5.5
27	Barytes (natural)	.7
28	Barytes (precipitated)	1.8
29	Calcium carbonate (whiting)	0.
30	Calcium carbonate (precipitated)	0.
31	Calcium sulphate (gypsum)	1.7
32	China clay (kaolin)	6.3
33	Asbestine (magnesium silicate)	5.1
34	American vermilion	10.
36	Lead chromate	7.7
39	Zinc chromate	9.5
40	Zinc and barium chromate	9.5
41	Chrome green (bluetone)	9.8
44	Prussian blue W. S.	9.
45	Prussian blue W. I.	8.5
48	Ultramarine blue	0.
49	Zinc and lead chromate	9.7
51	Magnetic black oxide	9.5
111	Brown (composite paint)	8.5
222	Black " "	8.8
3,333	White " "	4.5
444	Green " "	6.7
555	Black " "	8.2
666	Brown " "	7.7
777	White " "	7.2
888	Green " "	8.
2,000	1 coat zinc chromate, 1 coat iron oxide excluder	8.1
3,000	1 coat lead chromate	7.3
4,000	1 coat red lead, coat iron oxide excluder	7.7
100	Straight carbon black paint with turps and dryer	6.5
90	Straight lamp black paint with turps and dryer	5.7
5,555	Coal tar paint (over red lead)	5.2
1,000	Chrome resinate in oil	.7
	1 plate boiled linseed oil, 3 coats	1.5

TABLE III—SUMMARY OF BEST RATINGS IN ATLANTIC CITY PANEL PAINT TESTS, AFTER 33 MONTHS' EXPOSURE.

[List of ratings from 8 to 10, on scale of 10 = perfect.]

Panel.	Pigment.	Average.
34	American vermilion (basic chromate of lead)	10.
41	Chrome green	9.8
49	Lead-zinc chromate	9.7
39	Zinc chromate	9.5
40	Zinc and barium chromate	9.5
51	Black oxide of iron	9.5
4	Sublimed white lead	9
44	Prussian blue	9.
5	Sublimed blue lead	8.8
20	Willow charcoal	8.8
222	Composite paint	8.8
45	Prussian blue	8.5
111	Composition formula	8.5
9	Orange mineral	8.3
10	Red lead	8.3
555	Composite paint	8.2
12	Bright red oxide of iron	8.1
2,000	1 coat zinc chromate, 1coat iron oxide.	8.1
14	Venetian red	8.
888	Composite paint	8.

The Relation Between Human and Bovine Tuberculosis is the subject of the final report, just issued, of the British Royal Commission appointed to investigate this subject. The report is very extensive and discusses in much detail the results of experiments and studies lasting over seven years, but the main conclusion is that bovine tuberculosis is in some cases transmitted through infected milk, butter and meat to human beings, particularly children. The larger percentage of human cases investigated showed no signs of bovine tubercle bacilli, but some few of the cases showed unmistakable signs of the existence of the animal tuberculosis. As a result the commission strongly advises that "existing regulations and the supervision of milk production and meat preparation be not relaxed, and the Government should cause to be enforced food regulations planned to afford better security against the infection of human beings through the medium of articles of diet derived from tuberculous animals."

A River Training Wall at Rangoon, Burmah.

The city of Rangoon, Burmah, is located on the Rangoon River, about 40 miles from its mouth, and at this distance tidal influence exists. It is an important port of the Orient, and extensive works have been constructed during the past few years to improve the accommodation for ocean and river steamers. For the former there are wharves 2,830 ft. in length, with freight sheds, hydraulic cranes, railway sidings, and other equipment; for the latter there are 16 floating piers. The port has been threatened with destruction by the silting up of the river in front of the city and the scouring of a deep channel on the opposite side. It has been necessary therefore to take measures to restrain the river and to maintain deep water in front of the city. The situation is explained by the map, Fig. 1.

The river flows south as far as the city, but then makes a turn to the east to its junctionmark of 30 ft., and slopes of 1 on 2 for the outside and

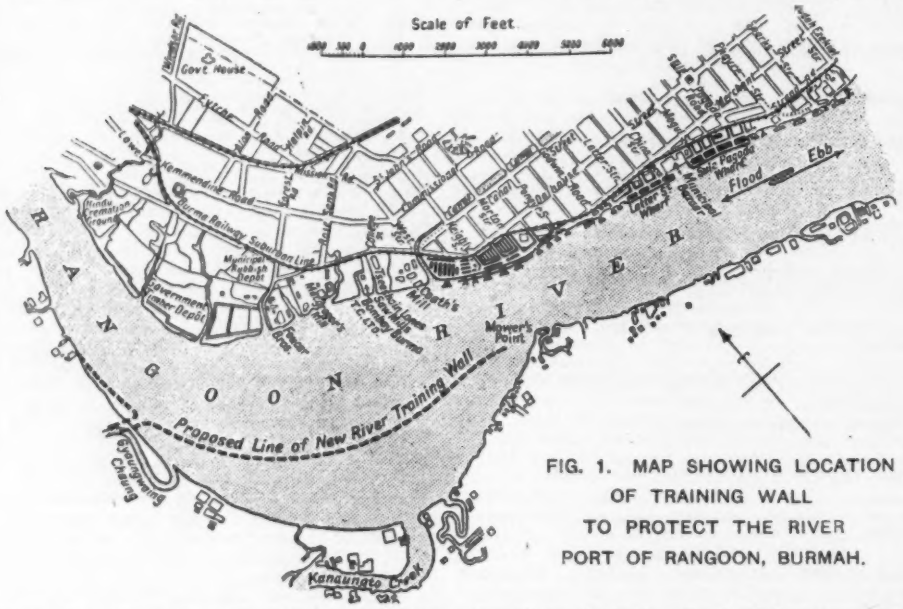


FIG. 1. MAP SHOWING LOCATION OF TRAINING WALL TO PROTECT THE RIVER PORT OF RANGOON, BURMAH.

with the Pegu River (just beyond the city), after which it again turns southward. In making this bend the river has gradually eroded the south bank (opposite Rangoon) so that the width has been increased to about 6,000 ft., while above the bend the width is about 2,000 ft. The action has been concentrated and aggravated by revetment work constructed by private parties above and below this concave bank. In addition to this, the swift current at ebb tide was diverted by the projection known as Mower's Point, so as to strike the north bank, thus tending to undermine the wharves and piers. Furthermore, the strong cross currents made it difficult to provide the necessary anchorages and moorings in the river.

The plan of improvement adopted was as follows. (1) to build a training wall* to check the flow of the current against the concave and eroded south bank at the bend; (2) to dredge a deep channel on the river side of this wall; and (3) to gradually fill and reclaim the space between the training wall and the river bank, using for this purpose the material obtained in dredging the channel. This plan was designed by Mr. G. C. Buchanan, M. Inst. C. E., Chairman and Chief Engineer of the Rangoon Port Commissioners, and the works are now approaching completion. The section of the river training wall is shown in Fig. 2. The following description is taken from an article in "The Engineer" (London), from which our cuts are reproduced:

*A description of the methods of quarrying the stone for this wall appeared in Eng. News, Dec. 8, 1910, p. 618.

The works now being carried out comprise: (1) The construction of a training wall 10,000 ft. long in depths varying from 20 ft. to 50 ft. of water below low water of ordinary spring tides; and (2) the dredging of a new channel simultaneously with the construction of the wall, the objects being to form a new channel inside the wall and to assist the reclamation of the bight outside the wall.

The method adopted for building the wall, a cross-section of which is given in Fig. 2, was as follows: In the first instance, a foundation course 230 ft. wide, composed of three rows of brushwood mattresses, is being laid. Each mattress is 125 ft. long, 75 ft. to 80 ft. broad and 3 ft. thick. The object of the mattresses is to form a carpet on the top of the silt to receive a loading of stone, and to assist in its speedy and even distribution over the area. The mattresses are built on slipways, launched and towed to the site, then loaded with granite deposited by hand from small barges and sunk. Subsequently steam hopper barges of 1,000 tons capacity dump granite on the mattresses until there is a thickness of 5 ft. of stone throughout.

On this foundation a random rubble half-tide training wall is being built. It is to have a width at low-water of 30 ft., and slopes of 1 on 2 for the outside and 1 on 1½ for the inside. The foundation course of mattresses and stone also forms a toe or apron to the wall, so that on erosion taking place in the vicinity of the new wall the apron sinks to the level of the river bed, and automatically revets the river bank and protects the wall from being undermined. The wall to within a few feet of low water is built up as explained, by dumping from the hopper barges, and from that point up to 5 ft. above low-water mark it is being deposited by hand.

When the foundation course is completed the shoal in front of the new wall will be removed by suction dredging, and the material thus obtained will be pumped into the bight on the other side of the wall. When the new channel is formed the wall will be gradually built up across the present channel. The end of the wall where the scour will be greatest is being built of granite blocks enclosed in wire crates. Ultimately, as circumstances permit, the land on the side of the wall away from the river (300 acres in extent) will be reclaimed and raised to high-water mark, a pitched slope being laid from the top to the new wall.

The first mattress was laid in November, 1909, and it was estimated that three years, dating from March, 1910, would be required to complete the foundation course, as laying such large mattresses in the middle of a tideway was a novelty, and attended with difficulties and risk. The work, however, has been pushed ahead vigorously, the difficulties have been overcome as they have arisen, and if nothing unforeseen occurs the whole of the mattresses will be laid and covered with stone by the end of June. The work then remaining will be comparatively simple, and will consist of dumping stone to form the wall and dredging the new channel.

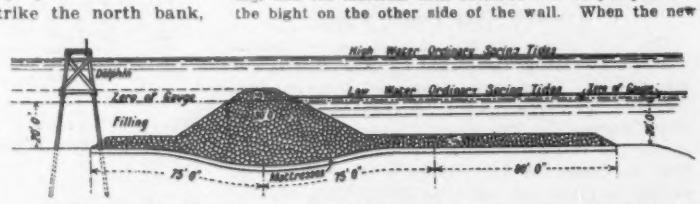


FIG. 2. CROSS SECTION OF TRAINING WALL IN THE RANGOON RIVER.

Reinforced-Concrete Tanks and Standpipes

A paper entitled "A New Theory for the Design of Reinforced-Concrete Reservoirs" was presented by Mr. H. B. Andrews before the Boston Society of Civil Engineers on Dec. 21, 1910. This paper with its very full discussion is published in the June, 1911, issue of the "Journal" of the Association of Engineering Societies," filling up some 45 pages of that publication. The matter is much too extensive for publication in Engineering News, but the original paper and much of the discussion is so valuable that we have prepared an abstract which is presented below:

A New Theory for the Design of Reinforced-Concrete Reservoirs.

By HIRAM B. ANDREWS.*

In circular water receptacles the water pressure at the base per square foot equals the product of the weight of water per cubic foot by the depth in feet. The tension in the wall per foot in height equals the product of the water pressure per square foot by the radius. For example, in the Manchester reservoir† (50 ft. diam., 72 ft. high) the water pressure per square foot at the base equals 62½ by 72 = 4,500 lbs. per sq. ft. The tension in the wall for the first foot in height equals 62½ by 71½ by 25, equal to 113,300 lbs. It has generally been assumed that this tension should be taken up entirely by steel reinforcement in the shape of horizontal steel rods bent to the radius of the reservoir and sufficiently lapped or mechanically attached to each other to develop the tensile strength when enveloped in concrete. It has been further assumed that a thickness of concrete sufficient to encase the steel reinforcement and to transmit the stress from one tension rod to another should be used. Empirical rules have been used for determining this thickness, relating only to obtaining the necessary bonding strength and a thickness of concrete supposedly enough to prevent seepage of water through the walls. No assumptions, to the writer's knowledge, have heretofore been made with the idea of utilizing the tensile strength of concrete prior to the construction of the reservoir at Rockland. The mixture of concrete that has generally been used in the past is one part of cement, two parts sand and four parts gravel or crushed stone, with the addition of hydrated lime or special compounds for densifiers, if I may use that term. As it is almost impossible to so thoroughly mix a 1:2:4 concrete either by hand or machine so as to make it entirely impermeable to water, the walls and floor have been usually coated with a cement mortar or with some other waterproofing compound.

As there is a tendency for the diameter of a reservoir to increase after it is filled with water due to the elasticity of the steel in tension, and as the base of the reservoir is practically rigid, due to its intimate contact with the foundation upon which it rests, it has been deemed necessary to install some reinforcing material extending from the base up into the walls to take care of the bending moment and shear at the base. The writer does not know, nor has he been able to find any exact method of obtaining, the amount of steel required here, but he thinks that in most cases heretofore it has been underestimated.

As the several operations of building up forms, placing steel and concrete preclude making the concrete work continuous, it is advisable to make as short intervals as possible between successive layers of concrete, and where these joints occurred to bond together the successive layers in the best possible manner. These instructions as outlined were followed out in the construction of the reservoirs at Waltham, Manchester and Lisbon Falls, except that the two latter reservoirs were made with a 1:1½:3 mixture of concrete plus 5% hydrated lime. For the Manchester reservoir the thickness of the wall at the base is 20 ins., and at the top 12 ins. The steel is designed for a unit working stress of 12,000 lbs. per sq. in. when the reservoir is full of water. At the base are installed rods which are embedded in the floor and turned up into the walls. These rods are 1 in. in diameter and are laid 12 ins. on centers and extend into the wall about 4 ft. 6 ins. The floor was finished with 1 in. granolithic, and the walls were plastered with two coats, making about 1 in. in thickness of 1:1 cement mortar. But after the filling of the reservoir various features were observed which showed that some improvement might be made in the design of future work.

*Engineer, Simpson Bros. Corporation.

†See references to the various structures noted on p. 124 of this issue.

There developed at several of the horizontal joints—especially at the three lower joints—between each day's work, some seepage of water, which in three places increased to positive leaks. The seepage, however, at the upper joints gradually stopped, presumably due to the filling of the pores by hydrated lime. We at first saw no reason why any leakage should develop. We had supposed that the concrete was rich enough, that the inside coating of 1:1 plaster, put on in such a manner as to make a double lap over the joints, and that the care taken in grooving and grouting the joints between each day's work, was sufficient to make the reservoir absolutely tight, and when we found that a little leakage did occur after taking all these precautions, we naturally began an investigation to determine the reasons therefore and to obtain for future work some protection against it. Upon examination of the reservoir after it was emptied, we found that there were horizontal cracks at the joints mentioned, about 30 ft. long, and extending through the wall, also that there were vertical cracks in the plastering extending upward for 20 ft. or so, and furthermore, that there were checks in the plastering from which the water oozed back into the reservoir after it was emptied.

From these observations we made the following deductions; that there was not enough vertical steel properly distributed to fully distribute the bending moment and shearing stress between the rigid base and the walls, and that the lack of this probably was the cause of the horizontal cracks. Second, that the ultimate strength of the concrete was probably exceeded when the reservoir was filled with water, thus producing the vertical cracks, and that these vertical cracks allowed the water to permeate into the walls and through them to the lines of least resistance, which would be the horizontal joints. Third, that the addition of a rich plaster coat with a more or less permeable concrete back of it was useless, as the usual crazing and the vertical cracking which would occur due to the tension would also allow the percolation of water through it.

This second deduction is the basis of this paper. From tests which have been made on concrete beams, it has been found that microscopic cracks have developed in the concrete on the tension side when the steel reinforcement was stressed to 4,000 or 5,000 lbs. per sq. in., or perhaps less; that these cracks gradually widened until they were finally visible. It is presumable that these cracks began when the ultimate strength of the concrete in tension was reached, and at a time that the unit stress in the steel was approximately ten times that in the concrete. These microscopic cracks were made visible by the application of water, and water lines following them could be seen. If these cracks developed in beams when the stress in the steel was only 5,000 lbs. or thereabouts, sufficient to admit water, why should they not develop in reservoir walls when the steel was stressed to the same point? And furthermore, if the walls were made so thin that they took little of the tensile stress, it being all thrown in the steel, at 12,000 or 16,000 lbs. per sq. in. or at any intermediate working value between these, why would not vertical cracks show in the walls, and permit water to seep into the walls?

One of the first requisitions in a concrete reservoir is to make it water-tight. The writer did not see how he could be sure that it would be water-tight if he so designed it, that the cracks in the concrete were predetermined. The only remedy seemed to be to make the walls so thick, and of such a composition, that their tensile strength would never be exceeded.

Having this in mind when we came to the design of the reservoir for Rockland, Mass., we decided to use an especially rich mixture of concrete, a thickness of wall which would insure that the ultimate tensile strength of the concrete would not be reached when the reservoir was filled, to use an increased amount of vertical reinforcement especially between the base and the walls, and to install a steel dam at each horizontal joint between each day's work to prevent any direct seepage of water through the joint, provided it entered it. The hydrated lime was omitted, as we considered the proposed density of the concrete did not require it for unpermeability, and also that where it had been used previously it had caused an unsightly efflorescence on the wall wherever there had been seepage. The plastering was omitted as we considered that any rigid coating upon this mixture of concrete was unnecessary, but instead, we applied three coats of soap and alum solution, commonly known as Sylvester Compound, to fill pinholes due to air bubbles, etc.

The thickness of wall at the base was determined as follows:

We assumed that a 1:1:2 concrete in tension was good for approximately 400 lbs. per sq. in., that the working stress in the steel, if by any chance the full tension was thrown upon it, would be 16,000 lbs. per sq. in., and that the ratio of moduli of elasticity between steel and concrete was 10, so that if the concrete was stressed to 300 lbs. per sq. in. the steel would be stressed to 3,000. The tension at the base of a standpipe 46 ft. in diameter and 104 ft. in height when filled with water would be $62\frac{1}{2} \times 104 \times 23 = 149{,}500$ lbs. per ft. in height. At 16,000 lbs. per sq. in. this would require 9.35 sq. ins. of steel per foot in height at the base, which was the section used. We made the thickness of wall 36 ins. at the base, the sectional area of concrete being $(36 \times 12) - 9.35$ equal to 422.65 sq. ins.

Let x equal unit stress in concrete, and $10x$ the unit stress in steel, then $422.65x + 9.35\ (10x) = 149{,}500$ lbs.; solving, x equals 290* lbs., which was considerably lower than the ultimate strength of 400 lbs. which we assumed. As we could find no tests of concrete in tension, we decided to have some made of large-sized briquettes at the Watertown Arsenal. We thereupon ordered our superintendent of construction on the Rockland job to make up some briquettes of special design, having a minimum cross section 4 ins. square. These briquettes were made of the same material and in the same manner as the concrete was made for our regular work, the 1:1:2 concrete being taken from the mixture while the regular work was going on.

We also made twelve 6-in. cubes for testing the compressive strength of the same concretes. The test pieces were made between Aug. 1 and Sept. 8, and the tests were made 60 days thereafter, between Oct. 29 and Nov. 7. The average tensile strength of the 1:2:4 concrete was 113 lbs. per sq. in., of the $1:1\frac{1}{2}:3$, 202 lbs. per sq. in., and of the 1:1:2, 281 lbs. per sq. in., there being no great variation from this average in any one of the tests. The average compressive strength of the cubes were as follows: 1:2:4, 2,280 lbs. per sq. in.; $1:1\frac{1}{2}:3$, 3,657; 1:1:2, 4,845. Upon examination of the fractured tension pieces it was evident that the increasing ratio of tensile strength when the richness of mixture was increased was due to the larger amount of mortar in cross section in the 1:1:2 concrete. As this concrete showed very few stones at the fractured section, while the 1:2:4 concrete showed a large number, it was plain that the adhesiveness of the mortar to the stones was not equal to the cohesiveness of the mortar.

In regard to the tensile specimens we quote from the paper made by the engineer in charge of the test:

> No change in the specimens was observed until rupture occurred. This took place quietly on a plane approximately perpendicular to the axis of the specimens and followed the surface of the gravel in nearly all cases.

From the results obtained we checked our previous assumptions. The average tensile strength of 1:1:2 specimens was 281 lbs. per sq. in. We assumed that in large-sized sections this possibly would be increased 25%, and that if these sections were reinforced by steel, that a further increase in strength of at least 10% might be expected. Our first assumption was made by reason of the fact that large-sized specimens in compression usually showed about this percentage of increase over small ones and our later assumption was based on the following theory:

If an unreinforced section is subjected to tension, when its ultimate strength is approached a crack will develop at the line of least resistance, and the fracture will always occur at this point.

If the section is reinforced, the ultimate fracture will not necessarily be at the point where the first crack developed, as the strain would be distributed nearly equally over the entire section by the reinforcing steel, and the fracture would take place at a plane in which was located the resultant of a number of weaker areas of concrete.

We, however, assumed our original figure of 281 lbs. increased by first 25% and then 10%, making a tensile strength of 386 lbs. per sq. in., which was probably nearly its actual value in the wall. It has been previously shown that the maximum tensile stress in the concrete would be 290* lbs. per sq. in., and the corresponding tensile strength in the steel 2,900* lbs., so that there would be a margin of about 30%* left for a factor of safety in the concrete without approaching the limitation on the steel. If, however, our assumptions had been wrong and no tensile stress whatever was taken by the concrete, which would be unreasonable, then we still had steel enough to take all the stress at 16,000 lbs. per sq. in. or its ordinary working value.

Applying this same line of reasoning to the Manchester reservoir, it is found that the tension at the base if divided proportionately between the steel and the concrete would be approximately 3,500 and 350 lbs. per sq. in., respectively. We have assumed that the ultimate tensile strength of the concrete plus the tensile be 202 + 25% + 10%, equal to 278 lbs. per sq. in., so that it is evident that the tensile strength of the concrete of the reservoir wall is exceeded, and that the vertical cracks developed might be expected.

We further find that at a point 25 ft. up, the tensile strength of the concreet plus the tensile stress in the steel just equals the stress due to water pressure at this point, and it was at about this place that the vertical cracks in the plastering disappeared.

There was never any trouble with the Lisbon Falls reservoir. A few damp spots appeared at first on the surface, but these soon disappeared. This reservoir is 62 ft. high and 50 ft. diameter, and the walls are 20-ins. thick at the base; $1:1\frac{1}{2}:3$ concrete was used and a 12,000-lb. per sq. in. working stress on the steel was assumed. The assumptions previously made would show a stress in the concrete of 235 lbs. per sq. in. and 2,350 lbs. per sq. in. in the steel. This stress in the concrete being less than its ultimate strength, we did not expect to find any vertical cracks with a resultant leakage.

The reservoir at Bridgewater is 78 ft. in height and 38 ft. diameter, and the walls at the base are 20 in. thick. This reservoir has never shown a particle of leakage. The stresses developed in concrete and steel are about 240 lbs. and 2,400 lbs. per sq. in. respectively. As an example of comparatively thin walls with a resultant leakage we will cite the reservoir at Attleboro, Mass. This reservoir is 100 ft. in height, 50 ft. in diameter, and the walls at the base are only 18 ins. thick; 1:2:4 concrete was used in its construction; 13,500 lbs. per sq. in. was the working stress adopted for the steel. From our previous examples, the stress in concrete and steel would be about 500 and 5,000 lbs. per sq. in., but the stress in the concrete so far exceeded its ultimate strength that numerous vertical tension cracks must have developed to a considerable height. In this case presumably the relative values of concrete and steel would not hold, but a stress of steel must have been developed to somewhere near the working value assumed. Several attempts have been made to make this reservoir water-tight, but each refilling has always resulted in leaks. The cracks in the concrete have probably been reproduced in any surface coating which has been applied.

A reservoir 70 ft. diameter and 22 ft. in height containing 20 ft. of water was constructed in Bondsville in 1908. The walls were 12 ins. thick, and composed of $1:2\frac{1}{2}:4\frac{1}{2}$ concrete with 5% hydrated lime added to the cement. The unit stress in the steel is 14,000 lbs. and in the concrete at the base 260 lbs. per sq. in. The ultimate strength of a $1:2\frac{1}{2}:4\frac{1}{2}$ concrete in tension may be assumed to be 100 + 25% + 10% = 137 lbs. per sq. in. Therefore the actual strength of the concrete was considerably exceeded. The owners expected an absolutely tight structure. When it was filled, considerable leakage developed at the joints, although there were no actual streams or jets of water coming through. The writer is informed that upon examination many vertical cracks were found in the concrete walls in the interior.

It is hard to say just what factor of safety against bursting there is in a concrete reservoir. We are, however, working on the side of safety in making the walls thick instead of thin. There is on record the case of the complete failure of a concrete reservoir in New South Wales described in Engineering News, of June 16, 1910. This reservoir is 40 ft. high and 40 ft. interior diameter, capacity 314,000 gals. The walls are $10\frac{1}{2}$ ins. thick at the bottom and $4\frac{1}{2}$ ins. thick at the top. It was built of a 1:2:2 concrete. The reinforcing rods are round rods with a working stress of

*The original paper gives this figure as 284 lbs., but the equation shows it to be 290 lbs.—Ed. Eng. News.

*Original paper gives these values as 274 and 2,740 lbs. and 40%, but see previous note.—Ed. Eng. News.

16,000 lbs. per sq. in. These rods are laid in one vertical plane.

Discussion.

MR. L. C. WASON.*—The following list of structures is supposed to be a complete list of all so far built. Where the word "standpipe" is used, it is used to mean a receptacle primarily used in connection with a water-works system. Where the word "tank" is used, it is understood to mean a receptacle which is not primarily for water-works, and may be made to contain either pure or impure water. These two words are arbitrarily used with this understanding in this tabulation.

1897. Standpipe. Germany. 132,000 Gals.
No description given.
"Das Wasserwerk der Stadt Calbe," by Paul Möller. "Zeitschrift des Vereines Deutscher Ingenieure, Vol. 41, p. 301 (March 13, 1897).

1899. Standpipe. Little Falls, N. J. 25,260 Gals.
10 ft. diam., 43 ft. high, walls 15 ins. thick at bottom and 10 ins. at top.
Transactions American Society of Civil Engineers, by Wm. B. Fuller, Vol. 50, p. 454.
Discussion of paper 954, June, 1905.
See also Transactions American Society of Civil Engineers, Vol. 54, Part E, pp. 433-34.

1902. Standpipe. Amsterdam, Holland.
"Bauwerke," "Beton und Eisen," Vol. 1 and 2, p. 9.

1903. Standpipe. Milford, Ohio. 93,000 Gals.
81 ft. high, 15½ ft. outside diam., walls 5 ins.-9 ins. thick. "Engineering News," Vol. 51, p. 184 (Feb. 25, 1904). "Engineering Record," Vol. 49, p. 382 (March 26, 1904). "Municipal Journal and Engineer," Vol. 21, p. 543. "Transactions" American Society of Civil Engineers, Vol. 54, Part E, p. 433.

Standpipe. Fort Revere, Hull, Mass. 118,000 Gals.
Octagonal tower, 33 ft. diam. at base, 84 ft. high; tank, 20 ft. inside diam., 50 ft. high; wall 6 ins. thick at base, 3 ins. at top.
"Description Concrete Steel Water Tower and Standpipe, Fort Revere, Hull, Mass.," by Leonard S. Doten. "Journal of the New England Water Works Association," Vol. 19, p. 33 (March, 1905). Abstract "Engineering News," Vol. 52, p. 598 (Dec 29, 1904). "Engineering Record," Vol. 48, p. 218 (Aug. 22, 1903). "Municipal Journal and Engineer," Vol. 21, p. 543 (Dec. 5, 1906). "Transactions" American Society of Civil Engineers, Vol. 54, Part E, p. 433, Edwin Thacher. International Enginering Congress 1905, paper 70. "Proceedings" Illinois Society of Engineers and Surveyors, Vol. 22, pp. 33-44 (1907), by Charles B. Burdick. "Cement and Engineering News," Vol. 19, p. 138 (June, 1907).

Standpipe. Leibnitz, Germany. 95,000 Gals.
Tank stands on high reinforced concrete support. It is 13 ft. 2 ins. in diam., and 9 ft. 10 ins. deep.
"Beton und Eisen," Vols. 1 and 2, p. 311.

2 Standpipes. Ekatérinoslav, Russia. 52,800 Gals. and 26,400 Gals.
Two tanks, one above the other, 32 ft. 10 ins. diam. The larger one, 49 ft. from the ground; the second, 97 ft. from the ground.
"Beton und Eisen," Vol. 3, p. 152 (June, 1904).

Tank. 64,250 Gals.
Supporting tower of pure concrete 36 ft. 1 in. high.
"Zeitschrift des Vereines Deutscher Ingenieure," Vol. 49, p. 1257 (July 29, 1905).

1904. Standpipe. Attleboro, Mass. 1,500,000 Gals.
50 ft. diam., 100 ft. depth of water; total height 118 ft.; walls 18 ins. thick at base, 8 ins. at top.
"Engineering News," Vol. 57, p. 212 (Feb. 21, 1907). (Editorial, same issue.) "Present Condition of Attleboro Standpipe." "Engineering News," Vol. 62, p. 199 (Aug. 19, 1909). (Editorial, same issue.) "Engineering News," (letter), Vol. 62, p. 230 (Aug.-26, 1909).

1904. Tank. England. 345,000 Gals.
70 ft. diam., tower 60 ft. high.
"Transactions" American Society of Civil Engineers, Vol. 54, Part E, p. 548, by R. G. Gifford Read.

Standpipe. Paris, France. 110,000 Gals.
Octagonal tower 28 ft. 6 ins. in diam.; tank is 22 ft. deep, 39 ft. in diam.
"Concrete Water-Tower at La Villette, Paris," by Max de Nansouty. Minutes of Proceedings of the Institution of Civil Engineers, Vol. 159, p. 436 (1904-5). (Part 1.)

2 Standpipes. France. 118,500 Gals.
12 ft. 7 ins. diam., 45 ft. high.
"Château d'Eau de l'Intercommunale du Centre," by O. Amiras. "Beton und Eisen," Vol. 5, p. 198 (August, 1906).

Tank. Belgium. 18,700 Gals.
Tank is divided into two pipes, each having a diam. of 9 ft. 10 ins. and a water height of 16 ft. 5 ins.
"Der Wasserturm in Forest (Belgien)," by O. Amiras. "Beton und Eisen," Vol. 4, p. 26 (February, 1905).
"Le Château d'Eau de Forest." "Le Ciment," Vol. 10, p. 29 (February, 1905).

Tank. Germany. Capacity not given.
"Ein Wasserturm in Eisenbeton für das Königl. Gestüt Rohrenfeld in Bayern." "Deutsche Bauzeitung," Vol. 39, Supplement, p. 16 (1905).

1905. Standpipe. Italy. 13,200 Gals.
20 ft. 4 ins. high, diam. of outer shell 15 ft. 2 ins.
"Engineering Record," Vol. 53, p. 371 (March 17, 1906). "Beton und Eisen," Vol. 4, p. 296 (December, 1905).

Tank. Germany. 137,200 Gals.
34 ft. 6 ins. diam., 20 ft. high.
"Beton und Eisen," Vol. 5, p. 134 (May, 1906).

2 Tanks. Hampton, Va. 66,000 Gals.
26 ft. 6 ins. diam., 16 ft. high; walls 12 ins. thick.
"Transactions" American Society of Civil Engineers, Vol. 54, Part E, p. 433, by Edwin Thacher.

1906. Standpipe. Anaheim, Cal. 172,000 Gals.
30 ft. 3 ins. diam., clear interior depth 32 ft.; rests on twelve reinforced-concrete columns 60 ft. long.
"Engineering Record," Vol. 56, p. 203 (Aug. 24, 1907). "Concrete Water Tank," by Edward P. Bailey. "Municipal Journal and Engineer," Vol. 28, pp. 366, 527 (March 9, 1910; April 13, 1910).

Tank. Kansas City, Mo. 25,000 Gals.
On tower 20 ft. high above roof of warehouse. Tank has 6 ins. side and bottom and 3½-in. roof, all reinforced with expanded metal.
"A Concrete Warehouse with Concrete Doors and Water Tank." "Engineering News," Vol. 58, p. 82 (July 25, 1907).

Standpipe. Waltham, Mass. 2,000,000 Gals.
Diam. 100 ft. 37 ft. deep.

Tank. Germany. 30,200 Gals.
"Der Wasserturm in Hard-Fussach," by Mich. Heimbach. "Beton und Eisen," Vol. 6, p. 247 (October, 1907).

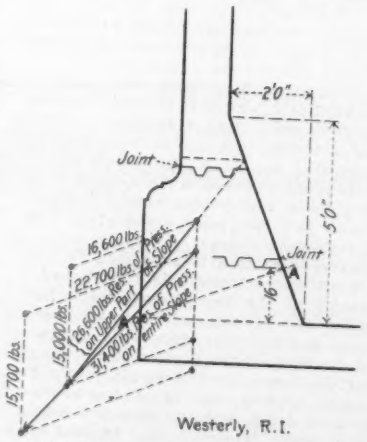
Tank. Shanghai, China. 90,000 Gals.
392 ft. high.
"Beton und Eisen," Vol. 6, p. 270 (November, 1907).

1907. Tank. Lincoln, Me. 108,000 Gals.
32 ft. diam., 16 ft. deep, 8-in. walls.

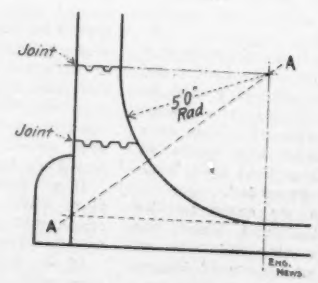
1908. 2 Tanks. Cuba. About 184,000 Gals.
34 ft. diam., 24 ft. 9 ins. high, supported on a tower 39 ft. high. Dome bottom 30 ft. diam., 5 ft. deep.
"Engineering News," Vol. 59, pp. 471, 571 (April 30, 1908).
"Discussion of the Design of Elevated Tanks and Standpipes," by Col. Wm. M. Black, U. S. A. "Transactions" American Society of Civil Engineers, Vol. 64, p. 539 (paper 1116; September, 1909).

Tank. Atlanta, Ga. 100,000 Gals.
Hollow, chimney-like shaft, 30 ft. inside diam. at the ground, and 22 ft. inside diam. at 90 ft. above, supporting a cylindrical tank.
"A Novel Reinforced Concrete Tower and Tank," by R. B. Tufts. "Engineering News," Vol. 61, p. 22 (Jan. 7, 1909). "Reinforced Concrete Water Tower for the Atlantic Compress Company, Atlanta," by R. B. Tufts. "Engineering Record," Vol. 59, p. 9 (Jan. 2, 1909).

Tank. California. Capacity not Given.
185 ft. high, 13 ft. outside diam., inside diam. 9 ft. to the 124-ft. level, at which point it is enlarged to 10 ft. 6 ins.



Westerly, R.I.



Attleboro, Mass.

JOINTS BETWEEN VERTICAL WALLS AND BASE IN REINFORCED-CONCRETE STANDPIPES AT WESTERLY, R. I. AND ATTLEBORO, MASS.

"A Unique Water Tower," by Edward P. Bailey. "Municipal Engineering," Vol. 36, p. 281 (May, 1909).

Standpipe. New Haven, Conn. 375,000 Gals.
50 ft. in diam., 25 ft. high; walls 18 ins. thick, reinforced with round steel bars.
"Mill Rock Water Tower, New Haven, Conn.," by Edward E. Minor. "Proceedings" of the Connecticut Society of Civil Engineers, 1908, p. 155. Abst[ract] "Engineering News," Vol. 59, p. 191 (Feb. 20, 1908).

Tank. Germany. 4[2],000 Gals.
37 ft. high.
"Beton und Eisen," Vol. 8, p. 2[1] (July, 1909).

1909. Standpipe. Empalme, Sonora, Me[x.]
30 ft. diam., 90 ft. high; wall [4]70,000 Gals. varies from 10 to 5 ins. [thi]ckness
"Engineering News," Vol. 62, p. [4]9 (Dec. 9, 1909). "Municipal Journal a[nd] Engineering," Vol. 28, p. 828 (June 8, 1910).

Standpipe. Brussels, Belgium. 28[0],000 Gals.
Tower and tank 145 ft. high.
"Cement Age," Vol. 9, p. 299 (N[o]vember, 1909), by H. Prime Kieffer.

2 Tanks. San Francisco, Cal. 32[,]000 Gals.
17 ft. outside diam., 20 ft. deep; 2½-in. walls, 14 ft. above ground, carried on 4-in. reinforced-concrete slabs.
"Engineering Record," Vol. 61, p. 726 (June 4, 1910).

Tank. Cajame, Sonora, Mexico. 60,000 Gals.
Dome supported bottom.
"Engineering Record," Vol. 61, p. 218 (Feb. 26, 1910).

Tank. Montview Station, Va. 100,000 Gals.
30 ft. diam.
"Engineering Record," Vol. 60, p. 447 (Oct. 16, 1909).

2 Tanks. Piombino, Italy. 330,000 Gals.
32 ft. 2 ins. diam., 30 ft. 1 in. high.
"Water Towers at Piombino," by Carlo Carvopassu. Preliminary and Interim Report of the Committee on Reinforced Concrete, Institution of Civil Engineers, p. 242 (June, 1910).

Standpipe. Bridgewater, Mass. 413,000 Gals.
Built on the State Farm; 30 ft. diam., 78 ft. high; walls 20 ins. thick at base, 12 ins. at top. Mix 1:1½:3½ of gravel. Inexperienced prison labor. Absolutely tight.

Standpipe. Manchester, Mass. 1,060,000 Gals.
50 ft. diam., 72 ft. high.
Standpipe. Lisbon Falls, Me. 913,000 Gals.
50 ft. diam., 62 ft. high.

1910. Standpipe. Westerly, R. I. 650,000 Gals.
40 ft. inside diam., 70 ft. high, 14-in. walls.
"Canadian Engineer," Vol. 19, p. 430 (Sept. 29, 1910). "Engineering Contracting," Vol. 34, p. 284 (Oct. 5, 1910).

Standpipe. Ixelles, Belgium. 212,000 Gals.
"Le Genie Civil," Vol. 56, pp. 445 (April 9, 1910).

Standpipe. Mittagong, New So. Wales. 314,000 Gals.
40 ft. diam., 40 ft. high; walls 10½ ins. thick at bottom, 4½ ins. at top.
"Engineering News," Vol. 63, p. 702 (June 16, 1910). (This tank failed.)

Standpipe. Rockland, Mass. 1,300,000 Gals.
46 ft. diam., 104 ft. high.

Standpipe. Cherry Valley, Mass. 200,000 Gals.
40 ft. diam., 21 ft. high.

Standpipe. Rochdale, Mass. 200,000 Gals.
40 ft. diam., 21 ft. high.

1911. Standpipe. Ashland, Mass. 300,000 Gals.
40 ft. diam., 32 ft. high.
"Handbuch für Eisen-betonbau," Vol. 5, p. 536, Ed. 2. Berlin, 1910. Wilhelm Ernst & Sohn. $4.40.
"Water Storage in Elevated Tanks and Standpipes," by H. E. Horton. "Journal of the Western Society of Engineers." Vol. 14, pp. 429, 432 (June, 1909).

The company with which I am associated has built two large standpipes, and a tank, the smaller 40 ft. in diameter and 70 ft. high at Westerly, R. I., and the larger at Attleboro, Mass., 50 ft. in diameter and 100 ft. high to water level.

From the experience with these standpipes I believe that the addition of considerable vertical reinforcement in the lower 8 or 10 ft. of the standpipe wall, the ends of which are turned out and bonded well into the floor, would assist very materially, and perhaps entirely obviate the formation of a horizontal crack in the lower few feet of a standpipe. It is obvious that under water pressure the walls, owing to the deformation of the steel, must increase in circumference, and the entire tank increase in diameter in proportion. At the bottom of the standpipe, however, owing to the rigid connection of the wall with the floor, this increase in diameter can not take place. This rigidity extends a short distance upward to the point where compression on

*President, The Aberthaw Construction Co., Boston, Mass.

floor changes to tension in wall. At the finish of a day's work near bottom of tank at or quite close to the plane of weakness above indicated there may be a direct movement outward of the wall above, relatively to that below, the joint when under pressure. When the pressure is relieved the elasticity of the hoops would tend to draw the wall back to its original position.

It is apparent then that unless precautions are taken, either by the use of very low stresses in the hoops or by the addition of vertical steel bars, there may be a distinct movement back and forth on some joint in the standpipe. That this movement is present is further indicated by the fact that when a standpipe has been kept full of water for some time, the leakage through these joints almost entirely disappears, but on emptying the standpipe and refilling it the little dams formed have apparently been broken down, as the leakage again occurs as vigorously as at first.

In the Westerly standpipe we endeavored to take care of this by increasing the amount of steel. At the lowest foot the stress was 6,000 lbs. per sq. in. This stress was increased 1,000 lbs. for each foot in height until we reached a maximum stress of about 12,500 lbs. per sq. in. No vertical steel, however, was used, and on filling the standpipe we found with the maximum head of water that a small line of dampness occurred in a horizontal joint about 3½ ft. above the floor of the tank. This damp place was nearly 30 ft. long.

By reference to the accompanying figure showing the section of the base in these two standpipes will be seen the position of what I call the critical joint. On the section of the Westerly standpipe a diagram is shown indicating the forces existing from the weight of the wall and the outward pressure of water on the sloping corner. The resultant of these two forces is shown. Projecting this resultant backward to the inside face of the wall the point is found where compression probably changes to tension. This is the critical point. It also comes very close to the joint where the second day's work left off. This is the joint that leaks. After the tank had been filled three weeks the leakage was just sufficient to dampen the surface but did not run down. If this point in a standpipe can be taken care of it is my belief that a standpipe can be made bottle tight.

In addition to the leak near the bottom of the westerly standpipe, three spots, and only three, appeared which leaked. One very close to the floor caused by a breakdown in the mixer resulting in a delay of an hour or so between batches of concrete, thus breaking the bond and destroying the monolithicity of the floor. There was another one about 23 ft. up, and one about 40 ft. These were grouted under high pressure and stopped. The worst of the three leaks before they were grouted dripped one drop in six seconds. Very great caution was exercised in finishing the joint between the days' work. Two men were kept on into the night after concrete was placed to scrub the top surface of concrete very carefully with brushes and water to get the laitance off, and also to wash the surface of all stone showing on top as clean as it was before being mixed, because cement bonds very well to clean aggregates; but it does not bond well to old cement. Roughing strips were put in the wall to form a groove. These were also removed by the men in the evening. Before concrete was placed the following day the surface was thoroughly washed with water, then coated with neat cement, which was rubbed in with a brush, and the grooves formed by the roughing strips were filled with neat cement.

The writer sees no objection from a purely theoretical standpoint, to making the wall of a standpipe so strong that the concrete in tension will resist the water pressure. In the application he thinks there are exceptions that could legitimately be taken to some of the assumptions, and that there are a number of practical objections applying to this method of design.

The author mentions reinforcing against the bending moment between the floor and wall. Inasmuch as there is a joint at this point between days' works, in this structure preventing any strength in the concrete mass, and inasmuch as it is difficult for the steel to get such a bond in the floor and wall as to resist a bending moment due to the great water pressure, it seems to the writer improper to apply this term to the stresses which here existed. The writer believes that the sole stress to be here cared for is that of shear, and believes with Mr. Andrews that it is wise to use a considerable amount of steel for this purpose. The writer has observed leakage through horizontal joints where there was no evidence of vertical cracks in the vicinity and believes that there is no relation between leakage of one set of cracks and the other.

Mr. Andrews, after determining by test the ultimate tensile strength of a specimen of concrete about 4 ins. square, makes the assumption that a larger mass is 25% stronger. The writer thinks it is unfortunate that the actual test upon which this assumption is based were not given, inasmuch as tension tests are rare and therefore are of very great value to engineers. Any such tests have entirely escaped the search of the writer. If, however, this assumption is based on compression tests, a different conclusion will be reached by examination of recent tests made at the Watertown Arsenal. It would appear that large specimens do not develop so high a unit stress as small specimens of the same proportion of materials.

It is unfortunate, also, that the tests upon which the assumption that reinforced concrete develops 10% greater strength than plain concrete are not given. If this is so it would be extremely valuable to the engineering profession to have the actual data. The investigation of Professor Talbot, as given in the recent edition of "Concrete; Plain and Reinforced," by Taylor and Thompson, gives the conclusion that reinforcing does not affect the tensile strength. Mr. Andrews states, "If the section is reinforced, the ultimate fracture will not necessarily be at the point where the first crack develops. . . ." In the writer's opinion, ultimate failure is where the first crack develops in his structure. An apparent increase of strength will be due to the steel solely. The writer's opinion, therefore, is that no greater stress can be theoretically assumed for tension in concrete than that obtained by actual tests of plain specimens and also that a suitable factor of safety should be allowed on this.

Assuming the theory to be worked out correctly, the following practical objections seem to the writer sufficient to throw serious doubt on the value of this method of designing standpipes. In each of those in which the writer was interested at one time during the placing of concrete an accident occurred delaying the work sufficiently to prevent its being truly monolithic. It is easy to make the inside face absolutely dense so as to be water-tight. However, it has been found quite difficult where two rings of steel have been used, and somewhat less difficult with a single ring to get absolute density around the bars. Any voids at this point reduce the cross-sectional area which is counted upon to resist tensile stress from water. There would also be an unequal stress on the inside and outside of a thick wall due to their distances from the center. The nature of these stresses makes it difficult to determine accurately their value, but it is probable that they would affect the uniformity of strength over the whole section somewhat. Moreover, shrinkage in a rich mixture of concrete setting in the air frequently produces cracks; and the larger the structure the more probable it is that cracks will actually exist. A single crack is sufficient to destroy the theory of the design. If the tank escapes cracks due to these causes when in service, it may receive them due to the difference in temperature in different seasons of the year between the outside and the inside. Finally, in actual tests made on beams where two identical specimens were made from the same batch of concrete, there was quite a wide difference in their strengths, and marked differences were noted between the strength of different batches. This irregularity, even when conditions are arranged for uniform and perfect results, is sufficient to throw doubt upon uniformity of the tensile strength of concrete throughout so large a structure.

By reference to the list of completed standpipes accompanying this discussion, it will be noted that several successful ones had thin walls, notably the one built years ago at Hull, Mass.

MR. F. A. BARBOUR.*—The new basis of design adopted by Mr. Andrews in which dependence is placed on the tensile resistance of the concrete, and the working stress of the steel so reduced that the resulting deformation will not crack the concrete, is of particular interest to designers of reinforced-concrete structures intended to hold water.

The general practice has been to stress the steel from 12,000 to 15,000 lbs. per sq. in., and to take no account of the tensile resistance of the concrete, building a wall thick enough to contain the metal and permit the placing of the concrete. Since the deformation of steel stressed in excess of about 4,500 lbs. per sq. in. exceeds the ultimate elongation of plain concrete, the construction of tanks in which the steel is worked at much higher stresses necessarily involves the cracking of concrete. The unknown factor in the problem has been as to the effect of the steel in developing a greater elongation in the reinforced concrete before final rupture than would be the case in plain concrete, or in distributing the deformation in the concrete into such a multiplicity of minute cracks that leakage would not result, or if it did result these minute cracks might be spanned by an elastic wash or coating. Mr. Andrews' conclusion that in the Attleboro tank vertical cracks must have developed is not only true but inevitable from the basis of the design. His assumption that such leakage as has occurred—which it may be stated is less than in any other standpipe which I have had opportunity to examine, except those at Bridgewater and Westerly, R. I.—results from these vertical cracks, is not a necessary corollary. From the most careful examination of the Attleboro tank there has been found no relation between the location of the leakage and these internal vertical cracks, as indicated by the water marks on the internal surface when the standpipe is drying out. It is true that in the critical section where the side walls, subject to stretch, join the bottom, there is a horizontal crack near which most of the leakage has developed, but even here, as appears from the work recently done in stopping these leaks, the explanation is found as much in imperfect concrete as in the existence of the horizontal crack.

Some months ago a successful attempt was made to stop the several leaks, then apparent, by grouting, and the method used may be of interest. On the assumption that the repairs should be made when the wall was stretched to the maximum limit, the work was done with the tank full. At the location of each leak a hole, larger on the inside, was cut into the wall exposing the steel. This hole was filled with small crushed stone, held in place by a piece of wire netting, and the whole plastered over with mortar, a short length of ¾-in. pipe passing through this plaster. Neat grout was mixed and poured into an 18-in. length of 6-in. wrought-iron pipe with flanged top and bottom. From the bottom a lead pipe connected with the pipe in the wall, and to the top a large carbonic acid tank under 3,000 lbs. pressure was connected. By a valve any pressure could be brought to bear on the grout, shooting it into the hole in the wall back of the hardened plaster facing. As much as ½-cu. ft. of grout was forced into the wall at one point, and through it for a distance of several feet where it again made its appearance at the surface. Obviously, the conditions found suggested that the leakage was here due more largely to imperfect placing of the concrete around the steel, which was too close together, than to the horizontal crack resulting from the deformation of the wall.

It is of interest to note that the steel when exposed was bright and clean.

In the Attleboro tank there have been from time to time a few small leaks, but at no time has more than 1% of the entire surface been even damp, a result that indicates that 1:2:4 concrete, properly placed, can be made watertight, although it may be better judgment to use a richer mixture.

It is not intended by the foregoing statement to assert that the deformation of concrete beyond its ultimate elongation is not the cause of leakage, but merely that in the Attleboro structure this result has not been indicated by the actual location of the leakage. That leakage increases with changing deformation is, however, made evident by the fact that the tank, if kept full, or at a nearly constant level, tends to become absolutely watertight. If, however, the tank is emptied and refilled, the leakage is always greater than before emptying—presumably the lime carbonate which had plugged the openings being crushed by the contraction of the steel as the pressure is removed. Again, it is of interest to note that there has been no leakage at the point where the gatehouse joins the standpipe. Whether this is because the gatehouse is on the shady side of the wall, or because it serves as an anchorage from which deformation starts, is perhaps debatable.

Returning to the basis of design adopted by Mr. Andrews there can be no question that if by

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working the concrete in tension the stresses in the steel can be kept below the point where the stretch of the wall will crack the concrete, one possible cause of leakage will be removed. It is to be noted, however, that if because of imperfect placing a weak point is developed and the concrete cracks, the steel will be deformed more than under the basis of design used in other tanks; and herein lies the risk. Further, it would seem that Mr. Andrews has failed to take into account the contraction of the concrete in setting, with the resulting condition that the concrete is already in tension and the steel in compression. How far this will reduce the effective strength of the concrete in resisting bursting stresses, and just how the two materials will work together in a relatively thick wall, must be demonstrated.

MR. SANFORD E. THOMPSON.*—The design of the Rockland reservoir is such a departure from previous methods of construction that a further word in regard to the methods of computation may be in order.

As the author of the paper has stated, the usual plan in the design of reinforced tanks or standpipes is to disregard the concrete except as a membrane for embedding the steel and furnishing a curtain to hold the water. The steel is designed, on the other hand, to take all the tension due to the water pressure. In Mr. Andrews' design the special feature is the fact that the concrete is designed to take a part of the tension. This, while a departure from common practice, is not a departure from theories which have been proved and accepted.

When a combination of steel and the concrete in which it is embedded is pulled, as in the bottom of any reinforced-concrete beam, they stretch together—provided the bars are small enough to give plenty of surface so as to prevent slipping—until the tensile strength of the concrete is reached. This is evidenced in numerous tests of beams made by various experimenters, which show conclusively that where the concrete adheres to the steel, the two act as one in the early part of the loading. In other words, these two materials stretch together, and as long as they are bonded together, they lengthen out uniformly. Now, from the principles of mechanics, within the elastic limits the stress equals the stretch in a unit length multiplied by the modulus of elasticity. Therefore, so long as the steel and the concrete stretch the same, on account of their bond, the stress in each material is inversely proportional to their modulus of elasticity and their area of section. Thus, if, at a certain load, the stress in the concrete is 284 lbs. per sq. in., its stretch with modulus of elasticity $E = 3{,}000{,}000$, will be $\frac{284}{3{,}000{,}000} = 0.000095$-in. per inch of length. In like manner, the stress in the steel is the same stretch, 0.000095-in., times its modulus of elasticity or since in steel, $E = 30{,}000{,}000$, the stress in steel is 30,000,000 times $0.000095 = 2{,}840$ lbs. per sq. in. This is irrespective of the areas of section of either the concrete or the steel.

The strength, however, is proportional to the areas, and hence if we have 423 sq. ins. of concrete and 9.4 sq. ins. of steel, the two together would resist (423 by 284) + (9.4 by 2,840) = 146,700 lbs.

The theory is the same as in columns having vertical reinforcement. The steel and the concrete act together in this way until the breaking strength of the concrete is reached, then the concrete breaks in a crack very fine at first, and the entire load is immediately given to the steel. It is evident, then, if we provide enough concrete so that its working strength in tension (together with the steel acting with it, as above) is not exceeded, no cracks should occur.

As a matter of fact, the reservoir, then, is designed like the tension side of a beam in the early stages of its loading before the minute cracks are formed. As a matter of safety, to be sure that the reservoir will not fail in case the concrete should crack from any unforeseen cause, enough steel is provided so that, even if it has to take all the tension or pull, the stress in it will not exceed 16,000 lbs. per sq. in. This is a higher allowable stress than is generally allowed in tank construction, because it is simply to guard against a contingency. In case it should be stressed to this amount, the reservoir would still be safe, although it would be apt to leak slightly at the point of crack. Under any conceivable conditions, therefore, it would appear that the reservoir would be safe.

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A Massive Concrete Statue.

By ROBERT H. MOULTON.*

Lorado Taft's statue of Blackhawk, the famous Indian chief of early Illinois fame, was unveiled at Oregon, Ill., on July 1, 1911. The statue is 48 ft. high, exclusive of the base, and represents a new departure in concrete construction, as applied to the art of real sculpture.

The statue of Blackhawk was three years in process of making. This estimate includes the time required in making plans, a model, and also a whole summer's work which had to be done over again when a cyclone blew away the first wooden structure used in connection with the modeling of the gigantic figure.

The exact site for the statue was determined by placing a rough 20-ft. model in different places on a lofty promontory, 250 ft. high, overlooking the Rock River, and then viewing it

Concrete Statue of Black Hawk Overlooking the Rock River Valley.

from many points. It was found, however, that the height and distances were so great that a statue of this size would be too small, and so it was decided to enlarge it to almost 50 ft., and to place it on the most prominent part of the cliff. As the statue now stands it is visible on a clear day from points 20 miles away.

Many difficult engineering problems had to be met and solved in building the statue. The first model was of plaster, 6 ft. high. This was enlarged by careful measurement to a frame of scantlings around a central shaft. When the whole figure had been framed in lumber the surface was made by stretching wire netting over the timbers. Over these forms burlap was fastened with nails. Later, the burlap was painted over with plaster of paris, giving the appearance of a plaster cast. Meanwhile the head was modeled in clay and cast by the usual process, the piece mold being saved for use again in casting the concrete. Next a 3-in. mold of plaster was placed around the entire figure. Over ten tons of plaster were required for this and many heavy timbers to support it. Finally everything was taken out of the mold and the entire space filled with concrete, only leaving a shaft 6 ft. in diameter running the whole length of the figure. An opening back of the statue allows a man to pass up the shaft and to look out of invisible windows placed at the top.

An assistant to Mr. Taft had the mechanical work in charge and a corps of three men were kept busy from the time the work was actually started until the completion of the statue. The concrete was poured about eight months ago, and the outer mold not removed until just before the unveiling, as it was desired that the material should have ample time in which to set. A motor-driven hoist was used in depositing the concrete.

The statue contains 2,275 cu. ft., and its total weight, including the foundations, is 336,770 lbs. The total maximum of wind pressure on the figure was figured by the consulting engineers to be 28,980 lbs., and the overturning effort of wind pressure, 673,785 lbs. The total moment of resistance to wind pressure acting through center of gravity on the 18-ft. base is 4,830,930 lbs. The base of the statue is of concrete, 18 ft. square and 4 ft. deep.

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Street Car Ventilation.*

By W. THORN.

The usual method of obtaining ventilation in street cars has been to provide a monitor deck in which are a number of small windows that can be opened and thus give a certain amount of ventilation. Nearly all cars have been built in this manner. The ventilation afforded by these small windows is largely by dilution; that is, such air as may enter these windows serves to freshen the air in the car. It is very doubtful, however, if this action extends far enough down into the car body to provide reasonably fresh air in the so-called "breathing zone." Furthermore, attempts to bring in enough air so that the fresh air is available in the breathing zone almost invariably result in strong drafts which are very objectionable to passengers.

The monitor window method of ventilation has been generally considered adequate. Studies made of this method of ventilation, however, show that it is almost impossible to accomplish good results without the presence of strong drafts and chilling the air in the car below the proper temperature.

The problem of providing adequate ventilation in a street car is not easy of solution. The difficulties arise from several causes, among which may be mentioned the comparatively small capacity of a car body in which many people are carried, also the various desires and whims of passengers as to proper ventilation, and the necessity for preventing drafts and chilling the air below the temperature required by the authorities.

The need for better ventilation than that afforded by the monitor or deck windows is much greater with modern prepayment cars than with the double-end type of cars. On the former the front platform is closed, while on the other cars it is open; therefore the air cannot circulate so freely through the prepayment type of car as through the double-entrance type when stops are made and the doors are opened to take on or discharge passengers.

To meet the need for better car ventilation several devices have been developed and are now on the market. The designers of some of these devices recognize that heating and ventilation should go together and have worked out their apparatus with this end in view. These ventilating devices or systems consist of two general types: (1) Those which depend upon the speed of the car for their operation, and all such devices work on the aspirator principle; (2) those which use mechanical means for circulation of the air and are practically independent of the speed of the car; mechanical systems of both the exhauster and blower principle have been brought out.

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The mechanical system* consists of a motor-driven exhaust fan located on the vestibule roof, an exhaust chamber formed above the ceiling of the car, openings located at various points in the ceiling, and intakes located at several points in the floor and connected to the electric heaters. The fan-motor set consists of a ¼-HP. motor direct connected to a specially designed 9-in. cone fan which will handle about 33,000 cu. ft. of air per hour under normal conditions of line voltage. The motor is connected direct to the 500-volt trolley circuit through a standard combination snap switch and fuse, and is started and stopped by means of this switch. The motor and fan are mounted in a suitable metal housing, which is connected to the exhaust chamber. The fan discharges through protected openings in each side of the housing.

The exhaust chamber in the upper part of the car is formed by dropping the ceiling about 4 ins. below the roof framing and is continuous from end to end of the car body. Communication between the car interior and the exhaust chamber is provided by 14 openings, each containing a circular adjustable register. By proper adjustment of the registers, a uniform velocity of air through all of them is obtained. The intakes are eight in number, four being located on each side of the car, under the seats, in such a manner as to be readily connected to the electric heaters. The connection between the screened opening through the car floor and the electric heater is made with a pressed metal duct. The size and number of the intakes are such as to permit of a maximum velocity of the air of about 400 ft. per minute, which is hardly perceptible to passengers.

The automatic systems installed are of several different kinds, but all depend upon aspirator action for their operation. One of these automatic systems, which has shown good results, comprises a number of "exhausters" located along each side of the monitor roof and attached to panels placed in the monitor or deck window openings. An opening in the panel communicates with the "exhauster." Intakes similar to those described in connection with the mechanical system are located in the floor and provide for a supply of fresh air. The "exhausters" are rectangular sheet metal boxes projecting outwardly from the panels to which they are secured, having openings top and bottom, and provided on the middle of each side face with V-shaped projections. The projections are placed horizontally on the faces of the exhausters and "split" the air into two streams, one flowing upward and the other downward. The air streams flow past the openings of the exhausters and by induction draw the air out from the car.

The impression has generally prevailed that thorough ventilation could not be effected without increasing the amount of energy used for the heating of the car. Experience with the deck window method of ventilation in which the heating and ventilating systems are not closely related is undoubtedly responsible for this impression. Tests conducted on street cars with ventilating systems, such as have been described, where the air in coming into the car passes first over the heating units and then is drawn up through the car and taken out at the top, demonstrate, however, that it is entirely possible to heat a car to substantially the same temperature with this system as when the deck window method is used, and with no greater expenditure of energy.

The reason for this lies in the greater efficiency of the heating units when working upon a continuous flow of fresh air passing over the heating surface as against the ordinary practice of placing the heating units in practically a dead air space where a considerable amount of the heating energy is lost in raising the temperature of surrounding parts of the car, such as the seats and the adjacent woodwork.

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working the concrete in tension the stresses in the steel can be kept below the point where the stretch of the wall will crack the concrete, one possible cause of leakage will be removed. It is to be noted, however, that if because of imperfect placing a weak point is developed and the concrete cracks, the steel will be deformed more than under the basis of design used in other tanks; and herein lies the risk. Further, it would seem that Mr. Andrews has failed to take into account the contraction of the concrete in setting, with the resulting condition that the concrete is already in tension and the steel in compression. How far this will reduce the effective strength of the concrete in resisting bursting stresses, and just how the two materials will work together in a relatively thick wall, must be demonstrated.

MR. SANFORD E. THOMPSON.*—The design of the Rockland reservoir is such a departure from previous methods of construction that a further word in regard to the methods of computation may be in order.

As the author of the paper has stated, the usual plan in the design of reinforced tanks or standpipes is to disregard the concrete except as a membrane for embedding the steel and furnishing a curtain to hold the water. The steel is designed, on the other hand, to take all the tension due to the water pressure. In Mr. Andrews' design the special feature is the fact that the concrete is designed to take a part of the tension. This, while a departure from common practice, is not a departure from theories which have been proved and accepted.

When a combination of steel and the concrete in which it is embedded is pulled, as in the bottom of any reinforced-concrete beam, they stretch together—provided the bars are small enough to give plenty of surface so as to prevent slipping—until the tensile strength of the concrete is reached. This is evidenced in numerous tests of beams made by various experimenters, which show conclusively that where the concrete adheres to the steel, the two act as one in the early part of the loading. In other words, these two materials stretch together, and as long as they are bonded together, they lengthen out uniformly. Now, from the principles of mechanics, within the elastic limits the stress equals the stretch in a unit length multiplied by the modulus of elasticity. Therefore, so long as the steel and the concrete stretch the same, on account of their bond, the stress in each material is inversely proportional to their modulus of elasticity and their area of section. Thus, if, at a certain load, the stress in the concrete is 284 lbs. per sq. in., its stretch with modulus of elasticity $E = 3{,}000{,}000$, will be $\frac{284}{3{,}000{,}000} = 0.000095$-in. per inch of length. In like manner, the stress in the steel is the same stretch, 0.000095-in., times its modulus of elasticity or since in steel, $E = 30{,}000{,}000$, the stress in steel is 30,000,000 times $0.000095 = 2{,}840$ lbs. per sq. in. This is irrespective of the areas of section of either the concrete or the steel.

The strength, however, is proportional to the areas, and hence if we have 423 sq. ins. of concrete and 9.4 sq. ins. of steel, the two together would resist (423 by 284) + (9.4 by 2,840) = 146,700 lbs.

The theory is the same as in columns having vertical reinforcement. The steel and the concrete act together in this way until the breaking strength of the concrete is reached, then the concrete breaks in a crack very fine at first, and the entire load is immediately given to the steel. It is evident, then, if we provide enough concrete so that its working strength in tension (together with the steel acting with it, as above) is not exceeded, no cracks should occur.

As a matter of fact, the reservoir, then, is designed like the tension side of a beam in the early stages of its loading before the minute cracks are formed. As a matter of safety, to be sure that the reservoir will not fail in case the concrete should crack from any unforeseen cause, enough steel is provided so that, even if it has to take all the tension or pull, the stress in it will not exceed 16,000 lbs. per sq. in. This is a higher allowable stress than is generally allowed in tank construction, because it is simply to guard against a contingency. In case it should be stressed to this amount, the reservoir would still be safe, although it would be apt to leak slightly at the point of crack. Under any conceivable conditions, therefore, it would appear that the reservoir would be safe.

*Engineer, Newton Highlands, Mass.

A Massive Concrete Statue.

By ROBERT H. MOULTON.*

Lorado Taft's statue of Blackhawk, the famous Indian chief of early Illinois fame, was unveiled at Oregon, Ill., on July 1, 1911. The statue is 48 ft. high, exclusive of the base, and represents a new departure in concrete construction, as applied to the art of real sculpture.

The statue of Blackhawk was three years in process of making. This estimate includes the time required in making plans, a model, and also a whole summer's work which had to be done over again when a cyclone blew away the first wooden structure used in connection with the modeling of the gigantic figure.

The exact site for the statue was determined by placing a rough 20-ft. model in different places on a lofty promontory, 250 ft. high, overlooking the Rock River, and then viewing it from many points. It was found, however, that the height and distances were so great that a statue of this size would be too small, and so it was decided to enlarge it to almost 50 ft., and to place it on the most prominent part of the cliff. As the statue now stands it is visible on a clear day from points 20 miles away.

Concrete Statue of Rock River Valley.

Many difficult engineering problems had to be met and solved in building the statue. The first model was of plaster, 6 ft. high. This was enlarged by careful measurement to a frame of scantlings around a central shaft. When the whole figure had been framed in lumber the surface was made by stretching wire netting over the timbers. Over these forms burlap was fastened with nails. Later, the burlap was painted over with plaster of paris, giving the appearance of a plaster cast. Meanwhile the head was modeled in clay and cast by the usual process, the piece mold being saved for use again in casting the concrete. Next a 3-in. mold of plaster was placed around the entire figure. Over ten tons of plaster were required for this and many heavy timbers to support it. Finally everything was taken out of the mold and the entire space filled with concrete, only leaving a shaft 6 ft. in diameter running the whole length of the figure. An opening back of the statue allows a man to pass up the shaft and to look out of invisible windows placed at the top.

An assistant to Mr. Taft had the mechanical work in charge and a corps of three men were kept busy from the time the work was actually started until the completion of the statue. The concrete was poured about eight months ago, and the outer mold not removed until just before the unveiling, as it was desired that the material should have ample time in which to set. A motor-driven hoist was used in depositing the concrete.

The statue contains 2,275 cu. ft., and its total weight, including the foundations, is 236,770 lbs. The total maximum of wind pressure on the figure was figured by the consulting engineers to be 28,980 lbs., and the overturning effort of wind pressure, 673,785 lbs. The total moment of resistance to wind pressure acting through center of gravity on the 18-ft. base is 4,830,930 lbs. The base of the statue is of concrete, 18 ft. square and 4 ft. deep.

*365 East 59th St., Chicago, Ill.

vices have been developed and are now on the market. The designers of some of these devices recognize that heating and ventilation should go together and have worked out their apparatus with this end in view. These ventilating devices or systems consist of two general types: (1) Those which depend upon the speed of the car for their operation, and all such devices work on the aspirator principle; (2) those which use mechanical means for circulation of the air and are practically independent of the speed of the car; mechanical systems of both the exhauster and blower principle have been brought out.

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to spend about $75,000,000 in construction of new lines and reconstruction of existing lines and on the equipment for operation of the entire system. About 44 miles of double-track subway line will be included in the system when completed.

The Public Service Commission on July 21, in accordance with the above decision of the Board of Estimate, awarded contracts to the Bradley Construction Co. and to Charles H Peckworth for five sections of the Lexington Avenue subway, extending from 26th St. to 106th St. (except for the section between 67th and 79th Sts.) The five contracts represent a total outlay of $16,139,428.

This extension of the New York rapid-transit subway system is by far the largest engineering work, measured by total outlay involved, now in progress in the United States.

Settlement of a Bascule Bridge Pier at Fall River, Mass., on July 25, 1911, deranged the operation of the bridge and will require reconstruction of the pier. The Brightman St. bridge over the Taunton River, a two-leaf bascule, was built 1905-1909, being opened in October, 1909. It cost about $980,000. The masonry piers of the bascule rest on a pile foundation, but the piles under the east end did not reach rock. This east pier has settled more than a foot, unequally, the southerly end being about 20 ins. low and the northerly end 10 ins. This settlement was sudden, being first seen on the morning of July 20; by the following morning it had settled about ⅝-in. more, it is reported. The bridge was closed to traffic. However, the bascule is still operative, although the two leaves separated several inches at mid-channel. The only explanation for the accident which has been offered is that the piles under the pier failed.

The Commission Plan of local government was adopted by the city of Passaic, N. J., on July 25 by a vote of 1,792 to 861. Local newspapers, the city officials, political leaders and prominent citizens generally, favored the plan.

A Continuous Flight of 388 Miles was made at Brussels, Belgium, on July 17, by one Olieslagers with a monoplane flyer. The course was confined to the local aerodrome.

During an aeroplane race between St. Petersburg and Moscow, Russia, on July 25, a machine fell killing the passenger and badly injuring the pilot.

A Bond Issue of $19,500,000 for terminals for the New York State Barge Canal is proposed by a bill signed on July 25, by Gov. Dix. The proposition must be submitted to a referendum of the people at the general election next November. Of the total amount, $9,740,000 is to be spent at New York City, $2,800,000 at Buffalo, $700,000 at Rochester, $850,000 at Oswego, $1,450,000 at Syracuse and smaller amounts at other cities along the line of the Canal. With the $101,000,000 originally authorized for the Canal, and the $7,000,000 authorizd last year for the Cayuga and Seneca extension, the total amount to be spent on the Barge Canal will be brought by this latest bond issue to $127,500,000.

An Engine Duty of 195,000,000 ft. lbs. per 1,000 lbs. of steam is guaranteed for two new triple-expansion pumping engines for which the City of Pittsburg has awarded contracts. These will be for the new Aspinwall pumping station which, when completed, will house five 20,000,000-gal. pumping units. Only four will be installed at present. At one end of the station will be erected the two complete new units with steam cylinders 32, 60 and 94 ins. in diameter and 66-in. stroke. The water ends are designed to deliver 20,000,000 gals. per day against a head of 275 ft. when the engine takes steam at 160 lbs. gage and 100° F. superheat. At the opposite end of the building two Holly engines, now at the Montrose station, are to be erected with new water ends. Space will be left in the center of the building for the fifth unit. The new equipment will draw its supply from a suction line passing under the boiler room and running parallel to the engines, discharging into the Montrose rising main, which supplies the Troy Hill Reservoir. At some future date the station will deliver to the proposed new Cabbage Hill Reservoir. The contract for the new pumping equipment was given to the Allis-Chalmers Co., of Milwaukee, for about $3,000,000.

The engine guarantees are seen to greatly exceed present pumping-engine record performances which have been considered noteworthy. The St. Louis pumping engines, noted in Eng. News, Apr. 5, 1906, showed a duty of 181,068,600 ft. lbs. per 1,000 lbs. of steam; the Milwaukee engines, noted in Eng. News, Mar. 10, 1910, showed 175,400,000 ft. lbs. per 1,000 lbs. of steam; an engine at Nashville, Tenn., (Eng. News, Dec. 21, 1909) has a record of 184,700,000 ft. lbs. and one at Albany, N. Y., in May, 1909, gave 183,115,000 ft. lbs. on test (Eng. News, Feb. 3, 1910). These guarantees even rise above what is claimed to be the record engine duty for any service—that of a Nordberg air compressor at Painesdale, Mich., which is reported to have shown by test the high figures of 194,930,000 ft. lbs. per 1,000 lbs. of steam.

Small Single-Chambered City Councils have superseded the larger two-chambered governing bodies at Pittsburg, Pa., and Scranton, Pa., as a result of recent action by the Pennsylvania Legislature. The Pittsburg Council consists of nine and the Scranton of five or seven members. The new councils were appointed by the Governor to serve until Jan. 1, 1912, when they will be succeeded by members chosen at the November elections. The act fixes the salaries of the Councilmen at between $2,000 and $6,500, as may be fixed by local ordinance. The Mayor of each of the two cities continues in office.

Personals.

Mr. J. A. McFarland, of St. Louis, Mo., has been appointed Chemist and Engineer of Tests of the St. Louis & San Francisco R. R. with headquarters at St. Louis, Mo.

Mr. A. C. D. Blanchard, Assoc. M. Can. Soc. C. E., formerly Assistant Engineer, Main Drainage Works, Toronto, Ont., has been appointed City Engineer of Lethbridge, Alberta.

Mr. C. E. Carson, formerly Superintendent of the Chicago Great Western R. R. at Clarion, Iowa, has been appointed Superintendent of the Northern Division at St. Paul, Minn., succeeding Mr. G. W. Vanderslice, resigned.

Mr. A. A. McGregor, formerly Division Master Mechanic of the Louisville & Nashville R. R., at Etowah, Tenn., has been made Assistant Master Mechanic of the shops at Evansville, Ind., for the same railway, succeeding the late J. B. Huff.

Mr. P. H. Everett, of Lynn, Mass., formerly Resident Engineer with the Massachusetts Highway Commission, has been appointed Assistant Chief Engineer of the Los Angeles County Highway Commission, of California, at a salary of $2,700 per annum.

Mr. Walter G. Eliot, formerly an assistant engineer of the topographical bureau of the Borough of Queens, New York City, has been appointed Park Commissioner of the same borough. Mr. Eliot is a graduate of Columbia University School of Mines.

Mr. E. O. Griefenhagen, recently in charge of the engineering and technical work of the Efficiency Division of the City of Chicago, Ill., has accepted a position as the head of a newly established department of the firm of Arthur Young & Co., certified public accountants, 1315 Monadnock Block, Chicago, to handle efficiency work in engineering lines and general employment problems.

Messrs. John A. Bensel, M. Am. Soc. C. E., State Engineer; Calvin Tomkins, Commissioner of Docks, of New York City, and R. A. C. Smith, of New York City, have been appointed a state commission to investigate port conditions and pier extensions in New York Harbor, by Governor Dix, under a law recently passed by the New York state legislature.

Mr. Benjamin Deakin, M. I. Mech. E., M. I. E. E., for many years of the engineering staff of the City and County of London Electric Supply Corporations, and subsequently Engineer and Manager in Australia for the British Insulated and Helsby Cable Co., and the Electric Supply Co. of Victoria, has joined the firm of Horrocks, Roxburgh Propty. Ltd., of Melbourne, Australia.

Prof. Gardner S. Williams, M. Am. Soc. C. E., has resigned the chair of Civil, Hydraulic and Sanitary Engineering at the University of Michigan, and will devote his entire time to practice as a consulting engineer, with offices at Ann Arbor, Mich. Prof. Williams has been engaged in teaching engineering for the past 13 years, six years at Cornell University, and seven years at the University of Michigan.

Mr. Henry Holgate, Vice-President, Can. Soc. C. E., has retired from the firm of Ross & Holgate, Consulting Engineers, of Montreal, Que., and has become President of the Cedar Rapids Power Co. The business of the firm of Ross & Holgate will be carried on in the future under the name of R. A. Ross & Co. Mr. Holgate was chairman of the commission which investigated the Quebec Bridge failure.

Obituary.

Nathan Haas, of the firm of Haas & Hea, Mining Engineers of Spokane, Wash., was drowned near Porcupine, Ont., on July 12, while attempting to escape from the forest fires which have been raging in that district. He was a graduate of the Michigan College of Mines, class of 1898, and had been Superintendent and Manager of numerous mining properties in the Northwest. He was 36 years old.

Wilfred L. Cooper, Division Superintendent of the Pennsylvania R. R. at Bedford, Pa., and his son Wilfred L. Cooper, Jr., a rodman with the Pennsylvania R. R. at Harrisburg, Pa., were drowned in the Susquehanna River on July 22. They were on a camping trip and lost their lives by the upsetting of a canoe. The father was 50 years old and the son 21 years; the latter was a graduate of Cornell University in the class of 1910.

Dr. Hugh Pitcairn, for 25 years a prominent American railway official, died in Hamburg, Germany, on July 19. He was born in Scotland in 1845 and came to this country with his parents while an infant. In 1859 he entered the office of the Pennsylvania R. R. in Pittsburg, Pa., to learn telegraphy. In 1865, he was made Assistant Trainmaster at Harrisburg. Later he became Superintendent of the Susquehanna Division, and was for two years Assistant General Superintendent. He had charge of the Evansville, Henderson & Nashville R. R. during its construction for the Pennsylvania R. R. After the completion of the road he was made its General Superintendent. He continued in this position until the road was sold to the St. Louis & Southwestern R. R., when he accepted the position of Superintendent of the Pittsburg, Cincinnati & St. Louis R. R. at Pittsburg, Pa. This position he was compelled to resign in 1875 because of failing health. Thereafter he studied medicine in Europe and in Philadelphia and settled in Harrisburg, where he practiced for a number of years. Dr. Pitcairn was a brother of Robert Pitcairn, for many years a prominent official of the Pennsylvania R. R. He is survived by four sons, Hugh, a civil engineer in the Philippine Islands, Roy, of Chicago, Ill., Norman B., a supervisor of the Pennsylvania R. R., and Frank, of Philadelphia, Pa.

Engineering Societies.

COMING MEETINGS.

SOUTH DAKOTA ENGINEERING SOCIETY.
Aug. 23-26. Annual meeting at Deadwood, S. Dak. Secy., R. G. Culbertson, Mitchell, S. Dak.

TRAVELING ENGINEERS' ASSOCIATION.
Aug. 29-Sept. 1. Annual convention at Chicago, Ill. Secy., W. O. Thompson, N. Y. C. Car Shops, East Buffalo, N. Y.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Sept. 12-15. Annual meeting at St. Paul, Minn. Secy., Clarence R. George, Houston, Tex.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 12-15. Annual convention at Atlantic City, N. J. Secy., A. P. Dane, Boston & Maine R. R., Reading, Mass.

COLORADO ELECTRIC LIGHT, POWER AND RAILWAY ASSOCIATION.
Sept. 13-15. Annual convention at Glenwood Springs, Colo. Secy., F. D. Morris, P. O. Box 1052, Colorado Springs, Colo.

ASSOCIATION OF EDISON ILLUMINATING COMPANIES.
Sept. 19-21. Annual convention at Spring Lake, N. J. Secy., N. T. Wilcox, Lowell, Mass.

MICHIGAN GAS ASSOCIATION.
Sept. 20-22. Annual meeting at Detroit, Mich. Secy., Glenn R. Chamberlain, Grand Rapids Gas Light Co., Grand Rapids, Mich.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 25-27. Annual convention at Chicago, Ill. Secy., Preston S. Millar, 29 West 39th St., New York City.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Sept. 26-29. Annual convention at Grand Rapids, Mich. Secy., A. P. Folwell, 239 West 39th St., New York City.

Vermont Electrical Association.

The tenth annual meeting will be held at Lake Dunmore, Vt., Sept. 13-14, 1911. The secretary is Mr. A. B. Marsden, Manchester, Vt.

Engineers' Society of Western Pennsylvania.

A visit of inspection was made July 15 by 500 members and guests to the plant of the McClintic-Marshall Construction Co., at Rankin, Pa., where the first three of the steel lock-gates for the Panama Canal had been erected for examination by Government inspectors before being shipped to Colon. The lock gates were described in Eng. News, Sept. 16, 1909, p. 304.

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